STEVENS INSTITUTE OF TECHNOLOGY

TMDL DRAFT REPORT

Passaic River Pathogen TMDL

Sarath Chandra K Jagupilla
David A Vaccari
Robert Miskewitz
Richard I Hires
Tsan-Liang Su

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The document presents the results of a pathogen water quality sampling and modeling study in the lower Passaic River at Paterson, New Jersey. Regression and formulize modeling for concentration and temperature boundaries, stormwater management model (SWMM) and Infoworks SWMM for boundary flows, and water quality analysis and simulation program (WASP) for water quality modeling were used. Reducing stormwater by 90% has greater effect on seasonal geometric mean than eliminating SWOs. The MOS was computed such that there is a 95% confidence of compliance. A 80% rollback of the upstream boundary is required along with eliminating the CSOs and reducing the SWO concentration by 90% to meet the water quality standards with a 95% confidence of compliance.

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List of Acronyms and Variables

a – Multiplier for River Velocity
b – Exponent for River Velocity
c – Multiplier for River Depth
CSO – Combined sewer outfall
d – Exponent for River Depth
EC – Escherichia *Coli*, per 100 mL
h – River Depth in m
H_n – Rate of Surface Heat Exchange in W/m^2
K_{aw} – Coefficient of Surface Heat Exchange, $W/m^2/C$
K_{EC} – Escherichia *Coli* decay coefficient, day^{-1}
NJDEP – New Jersey Department of Environmental Protection
Q – River Discharge in m^3/s
Q_d – Flow at Dundee Dam, m^3/s
Q_l – Flow at Little Falls, m^3/s
SWO – Stormwater outfall
SWMM – Storm Water Management Model
T_e – Equilibrium Temperature in C
TMDL – Total Maximum Daily Load
T_w – Surface Water Temperature in C
v – River Velocity in m/s
WASP – Water Quality Analysis and Simulation Program
θ – Temperature Coefficient

I. Executive Summary

In fall 2008, the New Jersey Department of Environmental Protection contracted with Stevens Institute of Technology to develop a pathogen total maximum daily load (TMDL) for an 8.65 mile combined sewer overflow (CSO) impacted stretch in the lower Passaic River near Paterson, NJ. The project included a weather driven sampling plan to capture water quality conditions in both dry and wet conditions, and data analysis and water quality modeling components. It was also planned that most sampling events will be conducted during summer as previous data indicated highest concentrations are observed in warm season.

Two dry weather events and one wet weather event were completed in 2009. Two more wet weather events were completed in 2010. Since all events till 2010 had flows below 1000 cfs, the sampling plan was redesigned to capture high flow conditions to for the remaining events. Therefore, 4 one day events – one dry weather event and three wet weather events – in high flow conditions were planned and executed in year 2011. The water quality parameters measured included E coli, fecal coliform, enterococci, nitrate nitrogen, total Kjedahl nitrogen, total phosphorus, total suspended solids, total dissolved solids, turbidity, conductivity, pH, dissolved oxygen, and surface water temperature.

To develop the water quality model boundary flow and concentration time series needed to be developed. The stormwater flows were modeled using SWMM. Flow from Little Falls was used as the upstream boundary, precipitation data was obtained from Rutgers from Hawthorne, NJ site, and flow at Dundee dam was used for calibration. Combined sewer outfall flows were obtained from Passaic valley sewerage commissioner (PVSC) owned Infoworks SWMM model.

Boundary concentrations for Totowa (upstream boundary), Pennington, Molly Ann, and Goffle (tributaries), and stormwater were modeled using Formulize software. The predictor variables used in these models were flow at Little Falls, flow at Dundee Dam, and rate of change of flow at Little Falls, and rate of change of flow at Dundee Dam. Models were selected based on their applicability over a wide range of conditions and their relative simplicity. The concentrations were bounded to 10 EC per 100 mL and 1000,000 EC per 100 mL. The CSO concentration was obtained by mass balance with the sewage baseflow and stormwater. Raw sewage was assumed to have a concentration of 1000,000 EC per 100 mL. The *NSE* values for the formulize models were 0.68 for Totowa, 0.61 for Molly Ann, 0.81 for Pennington, 0.83 for Goffle, and 0.88 for storm water. The boundary temperatures were modeled using simple linear regression using air temperature as the predictor variable.

The simulations of concentrations and flows were then used as inputs in to the water quality analysis and simulation program (WASP). The entire dataset was divided into a calibration dataset (dry event 1, dry event 3, wet event 1, wet event 2, wet event 4 and wet event 5) and an independent validation dataset (dry event 2, wet event 3, and wet event 6). The calibration of the WASP model was done by varying the temperature and decay constants of bacteria. The calibration *NSE* value was 0.525 and validation *NSE* was 0.0.249 for the selected temperature constant of 1.07 and decay constant of 0.1. The overall NSE was 0.462. Upstream sites were better modeled than downstream sites, although all sites were modeled adequately.

The WASP models indicated that the standards were never met under any conditions over the entire stretch of the river. Reducing the SWO concentrations by 90% has a greater impact on reducing the GM than eliminating the CSOs. Also, even reducing SWO concentration by 90% and eliminating CSOs together did not result in water quality standards being met. The only way to meet water quality standards is to roll back the upstream boundary concentration by 75%.

The margin of safety is proposed such that there is a 95% confidence of compliance instead of the default 50% confidence of compliance. For a 95% confidence of compliance the upstream boundary concentration must be rolled back by 80%.

II. Introduction

A. Area of Interest

The New Jersey Department of Environmental Protection (NJDEP) has identified an 8.65 mile combined sewer overflow (CSO) stretch in the lower Passaic River as impaired for pathogens and contracted with Stevens Institute of Technology in 2008 to develop a total maximum daily load (TMDL) with a load allocation (LA), waste load allocation (WLA), and a margin of safety (MOS).

The river segment of concern has the upstream boundary located at site labeled Totowa (Figure II-1) and the downstream boundary was the Dundee Dam. The study area is dominated by urban land use (Figure II-2). The area also contains significant forested area especially towards the upstream and moderate amount of barren land. Also minor amounts of wetlands and agricultural lands are present. The river segment has a variety of sources discharging into the river (Figure II-3). Of particular concern are the CSOs spread over the entire stretch, but more densely populated towards the upstream end. The number of CSOs in this stretch of the river is 36. The contour map (Figure II-4) shows steeper slopes on the upstream end and milder slopes on the downstream end of the stretch. The watershed area of the river segment of interest is shown in Figure II-5 ((Jagupilla, 2009). From the figure it is apparent that most of the forested area contributes to the watershed of the segment between Totowa and Northwest sites. The remaining two segments have watersheds with predominantly urban land use.

B. Pollutants of Concern

The pollutants of concern for this study are pathogens. Their presence in the river is measured by *Escherichia* coli, a pathogen indicator in freshwaters. However, historically fecal coliform was used as an indicator for pathogens in freshwaters. Therefore, both fecal coliform and *Escherichia* coli data were collected.

In addition to the above two, other water quality te-measures such as temperature, dissolved oxygen (DO), pH, turbidity, conductivity, total Kjedahl nitrogen (TKN), nitrate-nitrogen, nitrite-nitrogen, total suspended solids (TSS), total dissolved solids (TDS) and total phosphorus were collected. Also Enterococcus was measured for select samples at upstream and downstream boundaries to develop a correlation between *E. coli* and Enterococcus to help in developing a TMDL in the downstream tidal section.

Presence of pathogens in primary contact recreation water could cause numerous gastrointestinal illnesses including vomiting, diarrhea with fever or a disabling condition, or stomachache or nausea accompanied by a fever. Other possible adverse health impacts include upper respiratory illnesses (sore throat, cough, runny nose, cold or fever), rash, eye ailments, ear ache, headache, or backache (USEPA, 1986).

C. Water Quality Target

The river of segment of interest in the lower Passaic is classified as FW2 waters, which are fresh waters not designated as FW1 or Pineland Waters.

The relevant standard (NJDEP, 2009) is "*E* coli levels shall not exceed a geometric mean of 126/100 mL or a single sample maximum of 235/100 mL." The same document further states, "The Department shall utilize a geometric mean to assess compliance with the bacterial quality indicators at N.J.A.C.7:9B-1.14(d) 1 ii-iii. The geometric mean shall be calculated using a minimum of five samples collected over a thirty-day period. The single sample maximum shall be used for beach notification in accordance with N.J.A.C. 8:26 and to identify where additional ambient water quality sampling is needed to calculate a geometric mean."

Historically, the highest concentrations were observed in the warmer months in this stretch of the river (Jagupilla, et al., 2005). Further, primary contact recreation in this geographical area occurs primarily in the warm seasons. Accordingly, the water quality target to compute the total maximum daily load and its allocations for the present study is taken as the geometric mean shall be less than or equal to 126/100 mL in all warm seasons. The warm season is defined as the period between May 15 and September 15.

D. Water Quality Assessment

This stretch of the Passaic River has been assessed as impaired for pathogens by (NJDEP, 2003) and (Jagupilla, et al., 2005) using data provided by the Passaic Valley Sewerage Commissioners (PVSC) and historical data from United States Geological Survey (USGS).

E. TMDL Approach and Monitoring

A total of 9 sampling events – 3 dry and 6 wet weather events – were executed at 18 sampling sites from July 17, 2009 to October 11, 2011. The events are summarized in Table II-1and Table II-2. The sampling events were conducted with teams of graduate and undergraduate students of Stevens Institute of Technology who were led by either a professor or a research engineer.

Dry weather events were defined as those events where there was no precipitation for 72 hours before taking the first sample. A typical dry event consisted of sampling at 9 locations (6 in-stream sampling sites and 3 tributaries). Three sampling teams, each responsible for three sampling locations, and one transportation team, responsible for delivering samples to the external laboratory, were set up. Nominal sampling times were 6 AM, 10 AM, and 1 PM. This was widest possible spread of times possible given the constraints of the external laboratory operating hours and the holding time of samples for various water quality parameters.

Wet weather events, except wet event 1, were triggered by the observation of a combined sewer overflow. Wet weather event 1 was triggered by a predicted rainfall of 0.5 inches in the first 24 hours of sampling. A background sample was collected before the day of the predicted rain. On the first day of rain four samples were collected between 6 AM and 1 PM. This was followed by two more days of sampling with two samples each as the river returned to background conditions.

Dry events 1 and 2 and wet events 1-3 were all conducted under low flow conditions i.e. flow below the median level of ~1100 CFS. Therefore, dry event 3 and wet events 4, 5 and 6 were designed as single day events to capture observations under high flow conditions.

Table II-1 – Details of Dry Events

Event #	Date	Average Flow at Little Falls (CFS)	Previous Precipitation (Inches)
Dry Event 1	July 17, 2009	321	July 12 – 0.16 July 11 – 0.16 July 8 - 0.04
Dry Event 2	August 18, 2009	440	Aug 13 – 0.16 Aug 12 – 0.16 Aug 10 – 0.76
Dry Event 3	March 28, 2011	2272	March 24 – 0.69 March 23 – 0.41 March 22 – 0.01 March 21 – 0.47

Table II-2 – Details of Wet Weather Events

Event #	Dates	Average Flow at Little Falls (CFS)	Flow Range (CFS)	Conditions
Wet Event 1	15-22 Oct, 2009	327	183-418	Less CSO, More SW
Wet Event 2	13-16 Jul, 2010	511	133-936	Profuse CSO and SW
Wet Event 3	15, 28-30 Sept, 2010	141*/74	44-646*/44-115	Less CSO and SW
Wet Event 4	18 May 2011	2564	2331-2714	Less CSO and SW
Wet Event 5	19 May 2011	4918	4611-5265	More CSO, Less SW
Wet Event 6	11 Oct 2011	1144	1122-1161	More CSO and SW

* Conditions including the last sample which was taken after an intense rain event and resultant high flow. Removing this sample shows how low the flow was for this event.

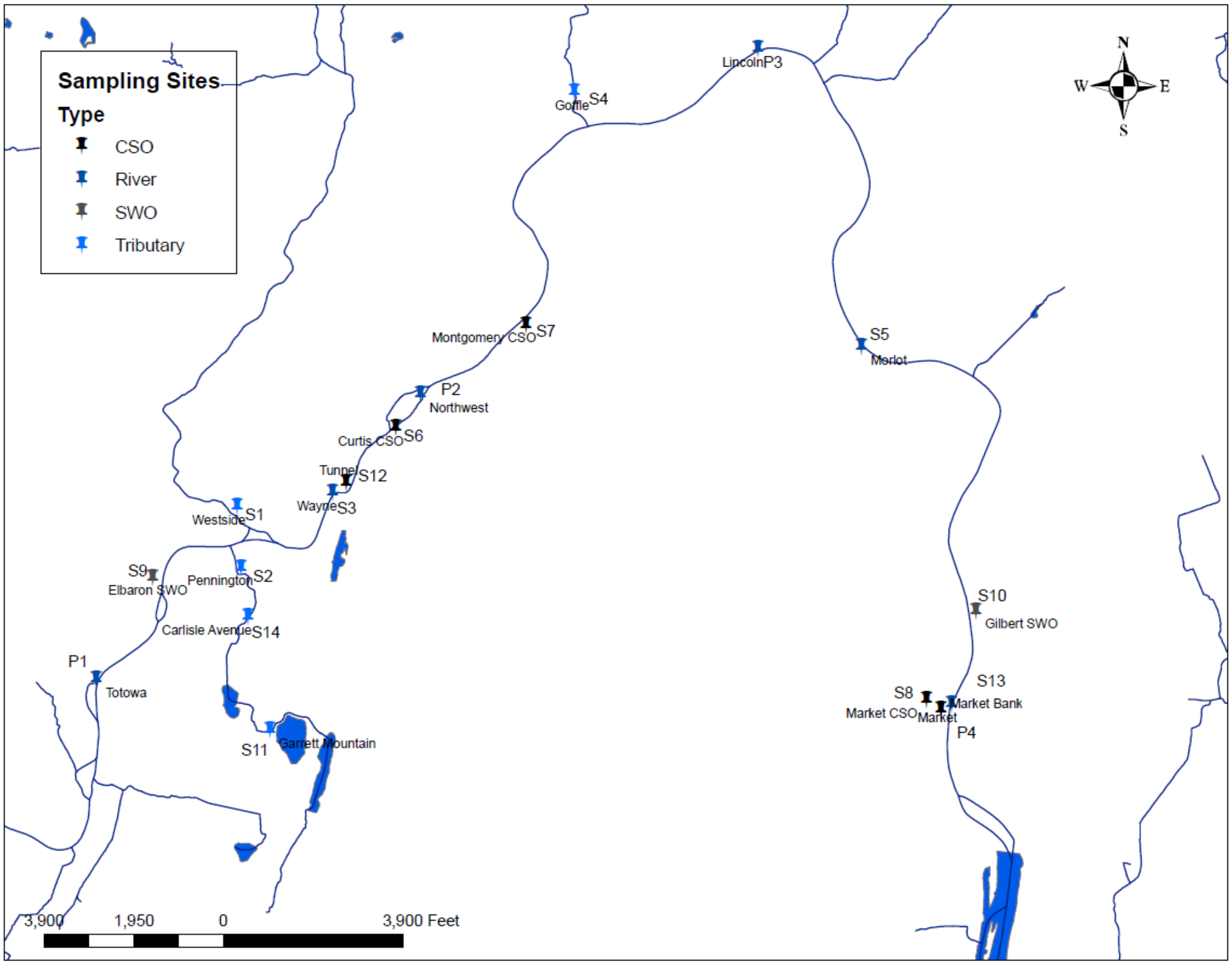

Figure II-1 - Study Area (Sampling Locations)

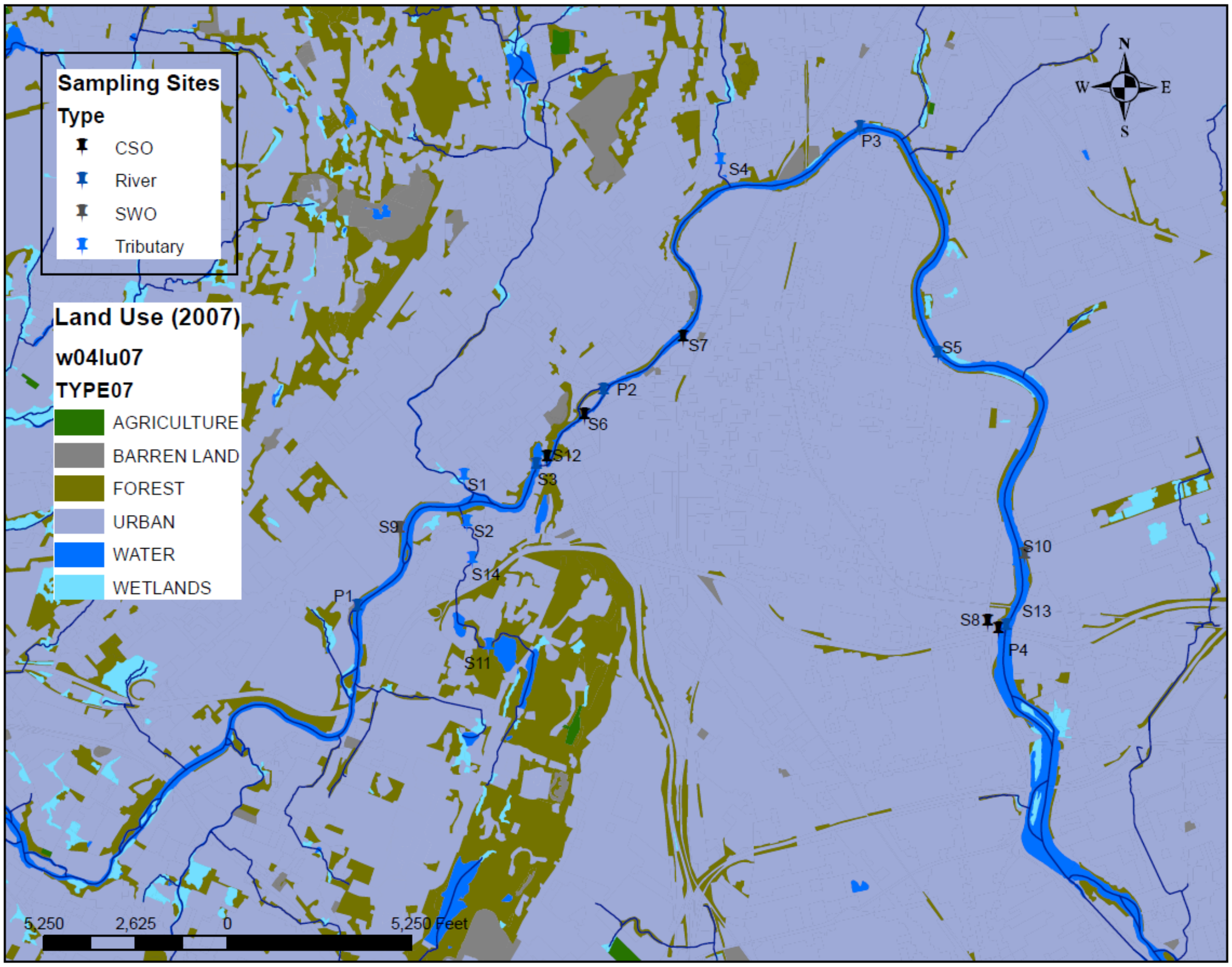

Figure II-2 - Study Area (Land Use)

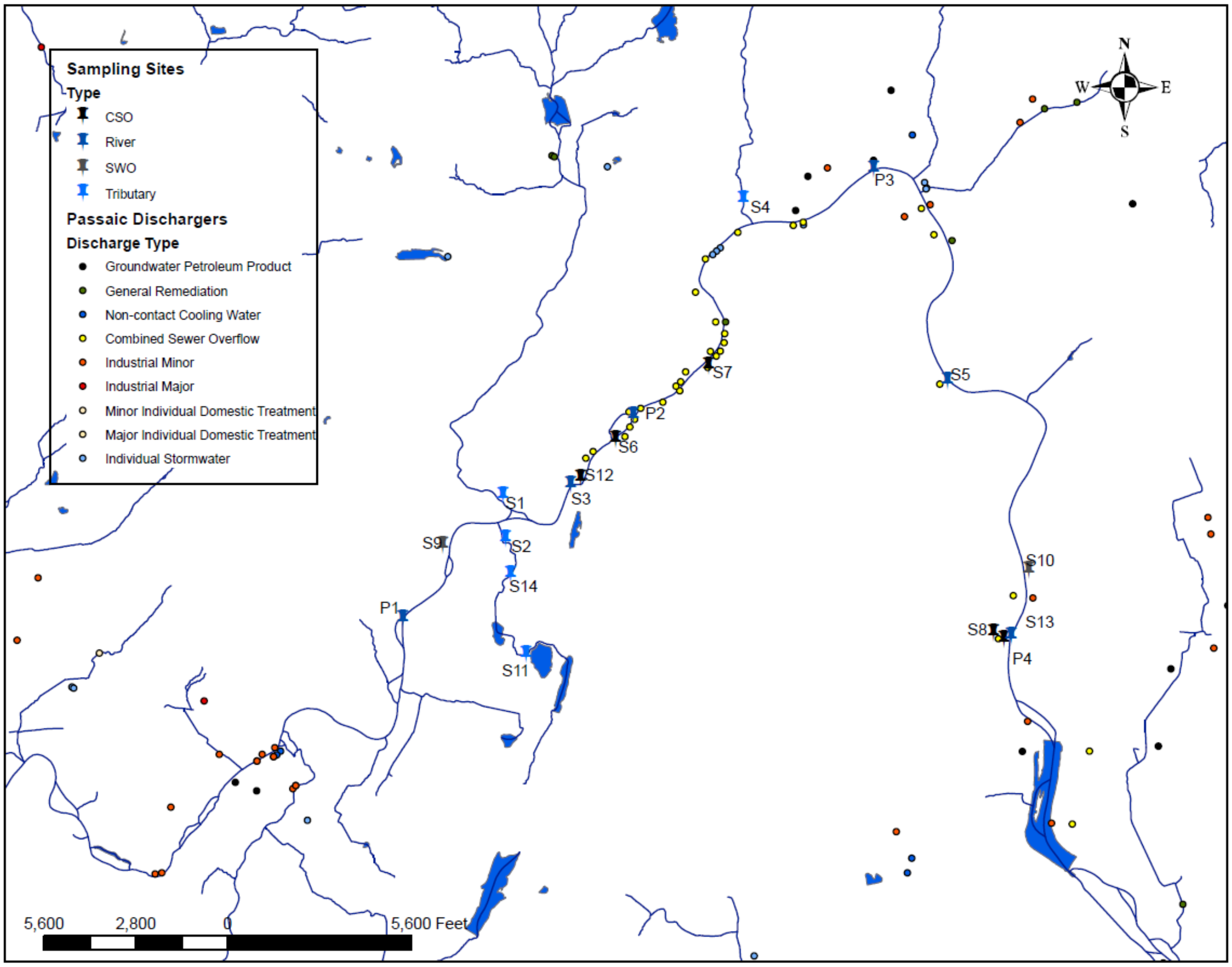

Figure II-3 – Study Area (Dischargers)

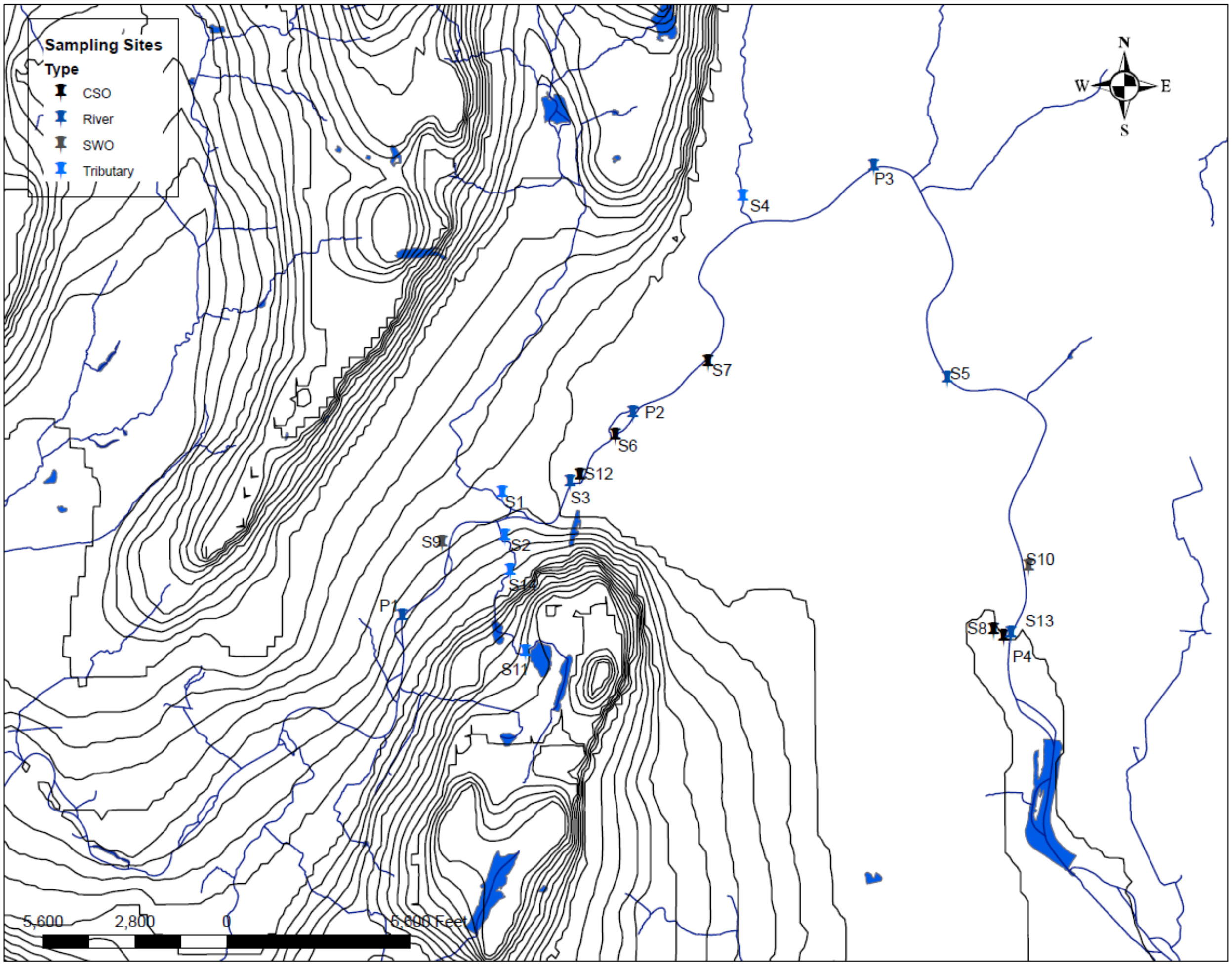

Figure II-4 – Study Area (Contour Map)

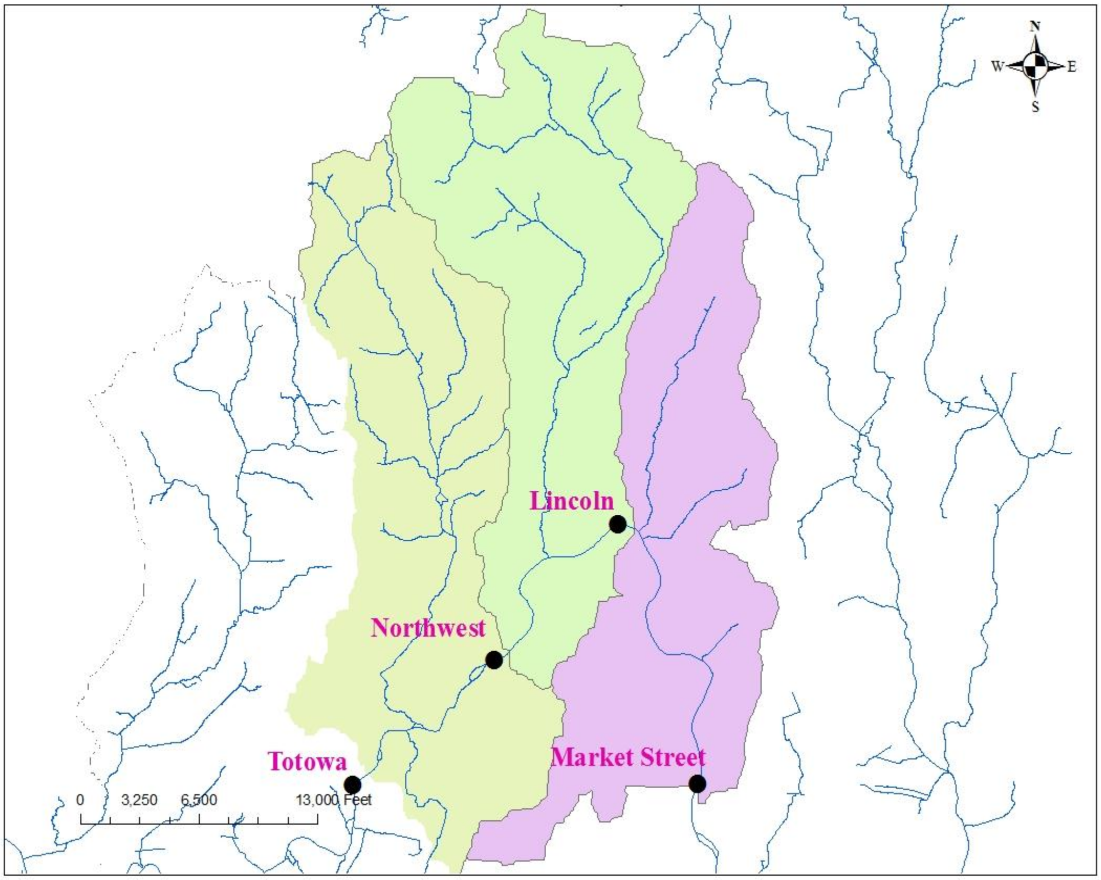

Figure II-5 – Study Area (Watershed)

III. Watershed Model Development

A. Spatial Extent and Relationship between Models

The data and models are related as shown in Figure III-1.

To simulate the bacterial indicator concentrations the WASP requires the following inputs:

1) Boundary flows
2) Boundary surface water E coli concentrations
3) Boundary surface water temperatures
4) Bacteria decay and temperature constants

The boundary flows that were included in the WASP model were the upland river flow which was obtained from USGS (site no. 1389500) at Little Falls, NJ, stormwater inflow obtained via a storm water management model (SWMM) simulation, and CSO inflow obtained via PVSC's existing SWMM InfoWorks collections system model. Hourly flow at Little Falls, NJ was obtained from the USGS online data archive for calendar years 2008, 2009, 2010, and 2011. The site of the USGS flow gauge is approximately 2.4 miles upstream of Totowa site. In order to account for inflows between the gauge and the upstream boundary, stormwater inflows were simulated for the area draining to the 2.4 miles upstream and added to the flow at Little Falls. Stormwater inflows to the WASP model were predicted using a SWMM model that encompassed the entire drainage area of the study area except for those areas served by the Paterson combined sewer system. Precipitation records used by the SWMM model were obtained from the Rutgers weather and climate network for the Hawthorne, NJ meteorological site (Rutgers University, 2012). The Hawthorne site is approximately 2.5 miles from Paterson where the study area is centered. CSO discharges were simulated by the PVSC InfoWorks model. This model was developed for PVSC by Hydroqual for the entire sewershed. The downstream boundary of the model was the Hamilton Avenue level gauge located approximately 0.5 miles downstream of the Market Street CSO. The Hawthorne precipitation record was used to simulate the CSO overflows.

Boundary E *coli* concentrations were modeled using Eureqa Nutonian software (Formulize, 2012) with flow at Little Falls, NJ and Dundee Dam, NJ (site no. 1389890) and the rates of change of flows as predictor variables. Simple linear regression was used to model boundary surface water temperatures based on air temperature data at Haworth, NJ (Rutgers University, 2012).

B. Simulation of SWO/Runoff Load Using SWMM

The quantity of stormwater outfall (SWO) discharge flowing into the WASP model domain was simulated using a SWMM model developed for this project. The SWMM model was similar in scale and detail to the PVSC Infoworks model (Hydroqual, 2003). It consisted of 27 subcatchments delineated using ArcHydro. Subcatchments located on the river drained directly to it while upstream subcatchments were routed through links that represented Molly Ann Brook, Pennington Brook, and Goffle Brook. Each subcatchment was characterized by area, SCS curve number, flow length, slope and percent imperviousness (Table III-1).

Tributary base flow was included in this model via a calculation of the dry weather flow increase from the upstream flow boundary (USGS Little Falls gauge) and the downstream flow boundary (Dundee Dam gauge). The flow difference was fairly consistent over the four year simulation period. As a result, a constant tributary base inflow rate was used.

Figure III-2 shows where the SWMM model output flows enter into the river.

C. Simulations of CSO Load Using InfoWorks

CSO discharges into the WASP model domain were simulated by the PVSC InfoWorks model. This model was developed for PVSC by Hydroqual for their entire sewershed. However for the current study only the Paterson area was used. Model development and calibration are detailed in Hydroqual (2003). The downstream boundary of the model was the Hamilton Avenue level gauge located approximately 0.5 miles downstream of the Market Street CSO. This level gauge is located close to a PVSC's Paterson flow meter. During the 2008 – 2011 project interval there are period where the level data was unavailable. For these periods a regression model was used to predict the level from the flow rate. The regression model resulted in an adequate fit (r^2 = 0.82) and had the equation:

$$level = 0.0497flow + 107.9974$$

The model was run to determine the flow of sewer water over the regulators at each CSO in the model domain. These flows were then routed to the WASP model cell at the appropriate location along the river.

D. WASP 7.4 – in-stream model

a. Hydrodynamic Simulation

The net flows option was used in WASP for the hydrodynamic simulation in this study. As there is a single flow direction, the results of both gross flows option and net flows option would be identical. There was not sufficient data or supporting documentation to consider using kinematic wave. Also, the data required to build a detailed hydrodynamic file did not exist.

b. Water Quality Simulation

The heat module (Wool, et al., 2008) was used to simulate both surface water temperature and E *coli* concentration in the river. The solar radiation data needed to model the surface water temperature was obtained from Haworth, NJ (Rutgers University, 2012).

The basic equation for modeling coliform bacteria decay from (Wool, et al., 2008),

$$\frac{\partial EC}{\partial t} = -K_{EC}\theta^{(T_w-20)}EC$$

Where

K_{EC} = coliform decay coefficient (day^{-1})

θ = temperature coefficient

T_w = water temperature in C

EC = Escherechia *coli* concentration (EC per 100 mL)

E. WASP Model Inputs and Assumptions

a. Simulation Period

The E coli concentrations were simulated for calendar years 2009, 2010, and 2011. The years 2009, 2010, and 2011 are the project sampling years, and the runs from these years were used for calibration and validation.

b. Node Positioning, Stream Network and Time Step

Figure III-3 shows the positioning of the nodes that separate the segments of the WASP model. Using the figure, and the street names for the CSO output from the PVSC Infoworks model, the inflows into each were determined (Table III-2). The segments that contain the in-stream sampling points are shaded in Table III-2.

c. Segment Length and Width

The segment length was set such that the numerical dispersion would be approximately equal to thelongitudinal dispersion. The longitudinal dispersion was calculated at selected sections of the river using dye studies performed on two different days (Table III-3).

The longitudinal dispersion coefficient was calculated from the dye study data using (Fischer, et al., 1979)

$$E_p = \frac{U^2(S_{t2}^2 - S_{t1}^2)}{2(\bar{t_2} - \bar{t_1})}$$

Where,

S_t – Temporal variance of the dye concentration (s^2)

U – Average river velocity (m/s)

$\bar{t}$ – Travel time (seconds)

E_p – Longitudinal dispersion coefficient (m^2/s)

The numerical dispersion is given by,

$$E_n = \frac{U\Delta x}{2}$$

Where,

E_n – Numerical dispersion (m^2/s)

Δx – Segment length (m)

The best modeling results are obtained when the segment length is chosen such that the resulting numerical dispersion is equal to the predicted longitudinal dispersion.

Based on the dispersion coefficients, for a velocity of 0.10 m/s, the required average segment length from Northwest to Lincoln must be at least 203 m and the average segment length from Northwest to Market must be at least 310 meters. Therefore, to be conservative an average segment length of 300 meters above Lincoln and an average segment length of 600 meters below Lincoln were chosen.

The segment width was measured using Google Earth. The selected segment lengths, and the measured widths from Google Earth, are summarized in Table III-4.

d. Depth and Velocity

The effect of river discharge on depth (*h*) was modeled using the relation,

$$h = cQ^d$$

Where *c* and *d* are empirical coefficients.

Velocity is simulated by dividing the flow by area of cross section. The area of cross section was computed in WASP assuming a rectangular cross section.

The depth multipliers and exponents were calculated by fitting a log-log model to field data of the USGS at Little Falls, Great Falls, and Dundee Dam (United States Geological Service, 2012) (Figure III-4, Figure III-5, and Figure III-6) to get an expected range for values in this stretch of the river.

Depth and width were measured on March 14, 2012 and June 14, 2012. The flow at Little Falls and Dundee Dam at the time of measurement was recorded (Table III-5 and Table III-6) provide the data collected on these two field trips. The flow at Little Falls at the time of first measurement was 12.21 m^3/s and at the time of second measurement was 32.57 m^3/s. The flow at Dundee Dam at the time of first measurement was 24.03 m^3/s and at the time of second measurement was 36.51 m^3/s.

The data results in an exponent of 0.32 and a multiplier of 0.64 for Totowa. The corresponding values were 0.19 and 0.60 for Lincoln. These values are in the vicinity of the values calculated using the USGS data (Figure III-4, Figure III-5, and Figure III-6).

The measured average depth at Market Street decreased with increase in flow. The decrease cannot be explained and is attributed to depth variability in measurement. Therefore, the depth is assumed to be constant. This results in an exponent of zero and the multiplier is then calculated as 2.39.

Table III-7 summarizes the depth multipliers and exponents that are used over the entire length of the river.

e. Flow Gauge Inputs

The flow data measured by the USGS gauge at Little Falls, NJ was used for the upstream boundary of the stretch. The flow data measured by the USGS gauge at Dundee Dam, NJ was used as the downstream boundary to calibrate the SWMM model outputs (United States Geological Service, 2012).

Table III-8 provides the geometric mean, median, minimum, and maximum flows for years 2008, 2009, 2010, and 2011 for Little Falls and Dundee Dam. The flow in 2008 is in between the flows in years 2009 and 2010 indicating its suitability as a TMDL year. The year 2011 had very high flows enabling the project team to execute sampling events in high flow conditions which was not possible during sampling years 2009 and 2010.

Figure III-7, Figure III-8, Figure III-9, and Figure III-10 are the time series plots of flows for the same years at Little Falls and Dundee Dam.

f. Meteorological Data

The precipitation data at Hawthorne, NJ (Rutgers University, 2012) was used as inputs to the SWMM and HydroQual models to simulate storm water, tributary and combined sewer flows. The air temperature, wind speed, dew point temperature, and solar radiation data were obtained from Rutgers weather station at Haworth, NJ (Rutgers University, 2012).

g. Boundary Input, CSO Loads, and SWO/Runoff Loads

Boundary conditions were needed to complete the water quality modeling. The temperatures and E coli concentrations were required at Totowa (the upstream boundary), the three tributaries (Pennington, Molly Ann, and Goffle), storm water, and combined sewer outfalls. The boundary inputs were modeled using Eureqa Nutonian Software (Formulize, 2012).

Multivariate polynomial regression (MPR) was earlier applied to the same stretch of river to make inferences about the location and behavior of pollutant sources (Jagupilla, et al., 2010). That work helped in the preliminary identification of sampling locations in the river before the start of sampling for this project.

The Eureqa software was used to generate nonlinear empirical models of the data collected from this project, as this software has the ability to test various transformations, such as logarithmic, exponential, Gaussian etc, of input variables along with exponents. Many of these transformations also provide reasonable bounds to the model simulations when the models are applied to data outside the fit dataset.

The *E* coli concentrations were log transformed to reduce the effect of extreme values on the regression. The input variables used in the model were discharge at Little Falls (Q_{lf}) in cubic feet per second (cfs), discharge at Dundee Dam (Q_{dd}) in cfs, rate of change of flow at Little Falls (dQ_{lf}/dt) in cfs per hour, and rate of change of flow at Dundee Dam (dQ_{dd}/dt) in cfs per hour. Due to less data, all data was used for fitting the models. Model selection was done using Occam's razor principle and behavior of models outside the fit dataset.

Totowa

The upstream boundary is possibly the most important boundary input for this stretch. Most of the other sources have relatively low flow rates and have minimal impact under most conditions. Further, the upstream boundary has a major impact on the final total maximum daily load computation and allocations.

Therefore, to ensure that a variety of conditions are well represented by the model, data collected by PVSC was also added to the data collected from this project to build the boundary model. It was observed that in general PVSC samples had significantly lower *E* coli concentrations than the samples collected in this project. This could be attributed to the designed nature of the sampling events where samples were taken in conditions that were biased towards higher concentration values.

However, PVSC did not collect *E coli* data. The *E coli* concentrations were simulated based on fecal coliform concentrations by using the linear models (Jagupilla, et al., 2012) developed as part of this project between fecal coliforms and E *coli*. The simulated E *coli* data was then added to the data collected as part of this project. This combined dataset was then used to build the boundary model. The model for Totowa, the upstream boundary, contained the flow at Little Falls and the rate of change of flow at Little Falls and Dundee Dam.

$$\log EC = 3.686 - 1.891 e^{-\frac{x^2}{2}}$$

$$where\ x = \frac{39.47 \times \frac{dQ_{lf}}{dt} - 473.8 \times \tanh(y)}{Q_{lf} - \frac{dQ_{lf}}{dt} - 1250 - 4.355 \frac{dQ_{dd}}{dt}}$$

$$where\ y = \frac{dQ_{lf}}{dt} - 673.1 \times \left(\frac{dQ_{dd}}{dt} \Big/ Q_{lf} \right) - 0.365 \qquad \text{III-1}$$

Figure III-11 shows the modeled *vs* predicted plot at this stretch for the data collected in this project. It could be observed that the entire range of values was modeled accurately, justifying the utility of the model to be used over the entire range of observations. The R^2 for this model was 0.68 for the entire dataset. There was no bias in the residuals.

Pennington

The model for Pennington site, which is on the Diamond Brook upstream of its confluence with the Passaic River, prominently featured the flow at Dundee Dam. The rate of change of flow at Dundee and the flow at Little Falls also were present in two terms.

$$\begin{aligned} \log EC &= 3.855 + \frac{49.90}{Q_{dd}} \\ &+ 1.023 \left(e^{-\frac{\left(10.24 - \frac{6413}{Q_{dd}}\right)^2}{2}} - e^{-\frac{\left(\frac{dQ_{dd}}{dt} - 7.072\right)^2}{2}} - e^{-\frac{(33.67 - 0.06709 Q_{dd})^2}{2}} - e^{-\frac{(0.1959 Q_{lf} - 66.47)^2}{2}} \right) \qquad \text{III-2} \\ &- 0.0002257 Q_{dd} \end{aligned}$$

The R^2 for the model was 0.81 and the residuals did not show any trend (Figure III-12).

Westside

The model for Westside site, which is on the Molly Ann brook upstream of its confluence with the Passaic River, did not contain the flow at Little Falls and the rate of change of flow at Dundee.

$$\log EC = 1.771 + 1.388 \log\left(12.72 + \frac{dQ_{lf}}{dt} \right) - 1.388 e^{-\left(\left(\frac{dQ_{lf}}{dt} + \frac{dQ_{dd}}{dt} \right)^2 \right)}$$

III-3

The R^2 for this site was a relatively high 0.61 (**Figure III-13**). The residuals did not show any bias.

The concentration in Molly Ann was adequately predicted just by the rates of change of flow at Dundee and Little Falls.

Goffle

The predicted concentrations at the Goffle site, which is on the Goffle Brook, increased with flow at Little Falls.

$$\log EC = 0.917 \times \ln\left(Q_{lf}\right) - 0.712 - 1.944 e^{-\left(\left(\frac{-7.915 - 0.652\frac{dQ_{lf}}{dt}}{Q_{dd} - 132.5}\right)^2\right)} \qquad \textbf{III-4}$$

The R^2 for this site was 0.83 (Figure III-14). The residuals had no bias for this model.

Stormwater

The stormwater concentration was modeled for the data collected at the Gilbert SWO site. Even though another stormwater site (Elberon SWO) was sampled, the data from this site was confounded with the in-stream concentrations in the Passaic River as the samples contained water from stormwater as well as the river. The model contained both the rates of change of flow, indicating the importance of precipitation intensity in modeling stormwater concentrations.

$$\log EC = 3.78 + \frac{0.0133\frac{dQ_{lf}}{dt} - 0.4753}{\tanh\left(\frac{dQ_{dd}}{dt} + 0.053 Q_{dd}\right)} \qquad \text{III-5}$$

The R^2 for the site was 0.88 (Figure III-15). The same model was used for the concentration of stormwater over the entire stretch of the river. The residuals were well behaved.

Combined sewer overflows

The concentration of raw sewage was assumed to be 10^6 EC per 100 mL. The range of E coli concentration in raw sewage is in the range of 10^4 to 10^9 EC per 100 mL (Clescerl, et al., 1999). (Vaccari, et al., 2006) estimates 10^8 total coliforms per 100 mL of raw sewage.

The concentration of the combined sewer overflow is calculated by mass balance as,

$$C_{CSO} = \frac{C_{SWO}Q_{SWO} + C_{SWG}Q_{SWG}}{Q_{CSO}}$$

Where

C – Concentration in EC per 100 mL

Q – Discharge in CFS

SWO – Stormwater Outfall

CSO – Combined Sewer Outfall

SWG – Sewage

h. Kinetic and Descriptive Parameters

The bacteria decay coefficient used in the WASP simulation of the river were determined by calibration. The data collected from the sampling events was divided into two sets – for calibration and validation purposes. The decay coefficient was determined using the calibration dataset and was then verified for acceptable performance on the independent validation dataset.

Calibration dataset – dry event 1, dry event 3, wet event 1, wet event 2, wet event 4, and wet event 5

Validation dataset – dry event 2, wet event 3, and wet event 6

The temperature coefficient generally recommended for bacteria is $\theta = 1.07$ (Chapra, 1997). Chapra (1997) recommends a decay coefficient of 0.8 per day for bacteria for freshwater. A range of decay rate coefficients from 0.1 to 0.8 were tested. The *NSE* values for the calibration dataset for decay coefficients 0.1, 0.3, 0.4, and 0.8 d^{-1-} are respectively 0.525, 0.524, 0.523, and 0.515. The validation *NSE* values were more sensitive with 0.249, 0.154, 0.104, -0.106.

Therefore, values of $\theta = 1.07$ and $K_{EC} = 0.1\ d^{-1}$ were used as the temperature coefficient and decay rate respectively because, however small the improvement in resulting fit, they have performed best on the calibration dataset.

F. Hydrodynamic Calibration and Validation

The SWMM was calibrated via comparison to the measured discharge over the Dundee Dam. Fit statistics for this model were calculated for the calendar year 2009. Model applicability was then verified for the calendar year 2010. The Nash-Sutcliffe Efficiency coefficients for the calibration period and verification period were 0.94 and 0.92 respectively.

Table III-9 provides the Nash Sutcliffe efficiency for the SWMM model for years 2008-2011. Poor results for 2011 are a result of extremely wet August and September when Paterson was flooded. The gauge measurements during this time are questionable.

G. Water Quality Calibration and Validation

The data collected from the sampling events was divided into two sets – for calibration and validation purposes. The decay rate was determined using the calibration dataset and was then verified for acceptable performance on an independent validation dataset. Figure III-17 provides the predicted vs observed comparison and predicted vs errors for the calibration dataset.

Figure III-18 provides the predicted vs observed comparison and predicted vs errors for the validation dataset. The *NSE* was 0.525 for the calibration dataset and 0.249 for the validation dataset. When all data, both calibration and validation, are combined, the overall *NSE* was 0.462 (**Figure III-18**).

Among the individual sites, the first three sites Wayne, Northwest, and Lincoln (**Figure III-19**, **Figure III-20**, and **Figure III-21**) are better predicted than the two downstream end sites Morlot and Market Street (**Figure III-22** and **Figure III-23**), although all are reasonably good.

H. Model Assumptions and Limitations

All formulize boundary models were based on data collected at flows below 1000 cfs. Therefore, application of the model over a wider range of flows should be done with caution.

Table III-1 – Subcatchment Characteristics

Subbasin	Sewer Type	Area (acres)	Imperviousness (pct.)	Flow Width (feet)	Slope (pct.)	Curve Number
39	Storm	2411	26.1	6650	1.7	74
40	Storm	1180.5	14.1	5875	3.3	77
41	Storm	1390.1	23.6	6900	2.4	75
42	Storm	1421.9	17.1	6500	0.5	71
43	Storm	800.9	38.3	6200	1.7	85
44	Storm	1337.7	33.9	5800	2.4	76
45	Storm	816.6	39.4	5041	1	80
46	Storm	1034.1	30.4	5550	1	75
47	Storm	1801.3	24.8	7700	3.5	78
48	Combined	350	40.3	2750	4.7	86
49	Combined	575.6	40.7	3830	5.1	83
50	Storm	867.7	32.1	2700	5.2	73
51	Storm	734	44.3	5750	0.8	83
52	Storm	1127	39	3500	1.2	80
53	Storm	388.3	41	1250	1.6	87
54	Combined	2563.1	49.1	3900	1	85
55	Combined	699.4	47.3	5250	1.3	85
56	Combined	837.4	22.8	5650	3.2	78
57	Storm	1260.6	49.4	6000	1.1	80
58	Storm	1059.3	31.6	4850	4.3	75
59	Storm	456	37.2	4225	3.9	77
60	Storm	923	39	5500	2.4	80
64	Storm	294.1	43.3	2600	1.5	79
65	Combined	353.8	56.1	1032	5	90
66	Combined	1079.4	51.1	2800	2.3	89
67	Combined	365.7	44.2	500	0.5	81
68	Combined	449.3	35.7	4500	1.5	75

Table III-2 – Inflows into each Segment

Segment Name	CSOs Flowing into Segment	SWOs Flowing into Segment	Tribs flowing into Segment
Totowa 1		Subcatch 58, Subcatch 57, Subcatch 59, Subcatch 60, Subcatch 68, Subcatch 50	
Totowa 2		Subcatch 67	
Totowa 3		Subcatch 56, Subcatch 49	
Elberon 1			
Elberon 2			Pennington (Diamond Brook),Westside (Molly ann Brook)
Pennington 1			
Westside 1			
Westside 2	SUM Park		
Wayne 1	Straight St.		
Tunnel 1	Market St. I/O, Market St.		
Tunnel 2	Curtis Pl. I/O, Curtis Pl. Mulberry ST., W.Broadway, NW St., Arch St.,		
Curtis 1	Bridge St		
Northwest 1			
Northwest 2	Montgomery St. I/O, Montgomery St., Hudson St		
Northwest 3	Keen St.		
Montgomery 1	Warren St., Bergen St.		
Montgomery 2	Short St.	Subcatch 48	

Montgomery 3	Sixth Ave		
Montgomery 4			
Montgomery 5			
Goffle 1			
Goffle 2		Subcatch 64	Goffle Brook
Goffle 3			
Goffle 4			
Lincoln 1			
Lincoln 2	2nd Ave	Subcatch 45	
Lincoln 3	3rd Ave	Subcatch 64, Subcatch 51	
Lincoln 4	4th Ave		
Morlot 1			
Morlot 2		Subcatch 52, Subcatch 53	
Morlot 3	E. 11th St., 33rd St.		
Morlot 4			
Gilbert 1			
Market 1	Bank St., 20th Ave		

Table III-3 – Dye Study Results

Site/Stretch	Date	Velocity (m/s)	Time of Travel (s)	Dispersion Coefficient (m^2/s)
Northwest – Lincoln	22 Oct 2010	0.17	21289	10.17
Northwest – Maple	26 May 2011	0.81	5123	10.82
Northwest – Market	26 May 2011	0.69	14676	15.53

Table III-4 – Segment Measurements

Segment	Length (m)	Width (m)
Totowa 1	300	65
Totowa 2	300	65
Totowa 3	300	65
Elberon 1	332	65
Elberon 2	332	65
Pennington 1	302	65
Westside 1	306	65
Westside 2	306	65
Wayne 1	158	24
Tunnel 1	284	20
Tunnel 2	284	20
Curtis 1	287	25
Northwest 1	284	30
Northwest 2	284	40
Northwest 3	284	45
Montgomery 1	336	55
Montgomery 2	336	60
Montgomery 3	336	60
Montgomery 4	336	60
Montgomery 5	336	60
Goffle 1	319	40
Goffle 2	319	40
Goffle 3	319	40
Goffle 4	319	40
Lincoln 1	629	40
Lincoln 2	629	55
Lincoln 3	629	60
Lincoln 4	629	60
Morlot 1	655	60
Morlot 2	655	60
Morlot 3	655	65
Morlot 4	655	80
Gilbert 1	615	85
Market 1	565	150
Market 2	565	150

Table III-5 – Depth and Section Width on March 14, 2012

Site	Section Width (m)	Width Percentiles	Depth (m)
Lincoln	82.91	25%	0.76
		50%	1.07
		75%	1.04
Totowa	91.44	25%	0.88
		50%	1.74
		75%	1.68
Market	109.12	25%	2.93
		50%	2.01
		75%	2.23

Table III-6 – Depth and Section Width on June 14, 2012

Site	Section Width (m)	Width Percentile	Depth (m)
Lincoln	87.48	25%	1.07
		50%	1.24
		75%	1.14
Totowa	102.41	25%	1.60
		50%	2.06
		75%	2.23
Market	109.42	25%	2.46
		50%	2.13
		75%	0.84

Table III-7 Depth Multiplier and Exponent corresponding the the segments in Table III-2

	Depth (m)	
	Multiplier	**Exponent**
Totowa 1	0.64	0.32
Totowa 2	0.64	0.32
Totowa 3	0.64	0.32
Elberon 1	0.64	0.32
Elberon 2	0.64	0.32
Pennington 1	0.64	0.32
Westside 1	0.64	0.32
Westside 2	0.25	0.54
Wayne 1	0.25	0.54
Tunnel 1	0.19	0.60
Tunnel 2	0.19	0.60
Curtis 1	0.19	0.60
Northwest 1	0.19	0.60
Northwest 2	0.19	0.60
Northwest 3	0.19	0.60
Montgomery 1	0.19	0.60
Montgomery 2	0.19	0.60
Montgomery 3	0.19	0.60
Montgomery 4	0.19	0.60
Montgomery 5	0.19	0.60
Goffle 1	0.19	0.60
Goffle 2	0.19	0.60
Goffle 3	0.19	0.60
Goffle 4	0.19	0.60
Lincoln 1	0.19	0.60
Lincoln 2	0.19	0.60
Lincoln 3	0.19	0.60
Lincoln 4	0.19	0.60
Morlot 1	0.19	0.60
Morlot 2	0.19	0.60
Morlot 3	0.19	0.60
Morlot 4	0.19	0.60
Gilbert 1	0.19	0.60
Market 1	2.39	0.00
Market 2	2.39	0.00

Table III-8 – Statistics for Flows at Little Falls and Dundee Dam

Statistic	Little Falls (CFS)				Dundee Dam (CFS)			
	2008	2009	2010	2011	2008	2009	2010	2011
Geometric Mean	733	845	566	1,619	821	896	704	1,805
Median	806	820	570	1,726	893	845	646	1,937
Minimum	67	168	18	138	31	29	56	34
Maximum	7,566	5,498	15,764	20,841	7,556	7,259	16,160	27,529

Table III-9 – Nash Sutcliffe Efficiency Coefficients for Various Years

Total Flow (cubic feet)	Year	Nash Sutcliffe Efficiency Coefficient	Dundee Dam	Little Falls	Watershed Contribution (cubic feet)	Predicted Runoff (cubic feet)	Predicted Runoff with Base Flow (cubic feet)
			Measured (cubic feet)				
	2008	0.53	4.37E+10	4.04E+10	3.23E+09	1.73E+09	3.08E+09
	2009	0.66	3.66E+10	3.47E+10	1.33E+09	1.34E+09	1.68E+09
	2010	0.43	4.54E+10	4.13E+10	2.94E+09	1.81E+09	2.80E+09
	2011	-0.15	9.12E+10	8.19E+10	6.76E+09	2.47E+09	4.40E+09

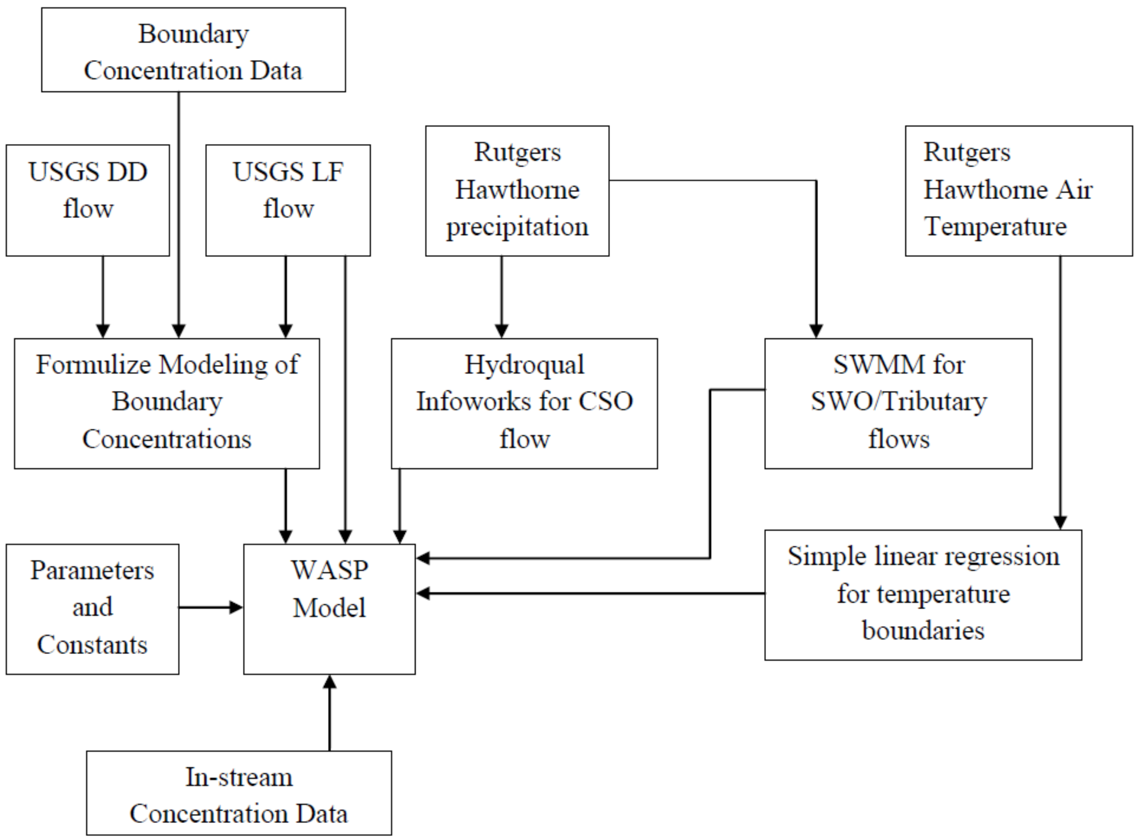

Figure III-1 – Relationship between Data and Models

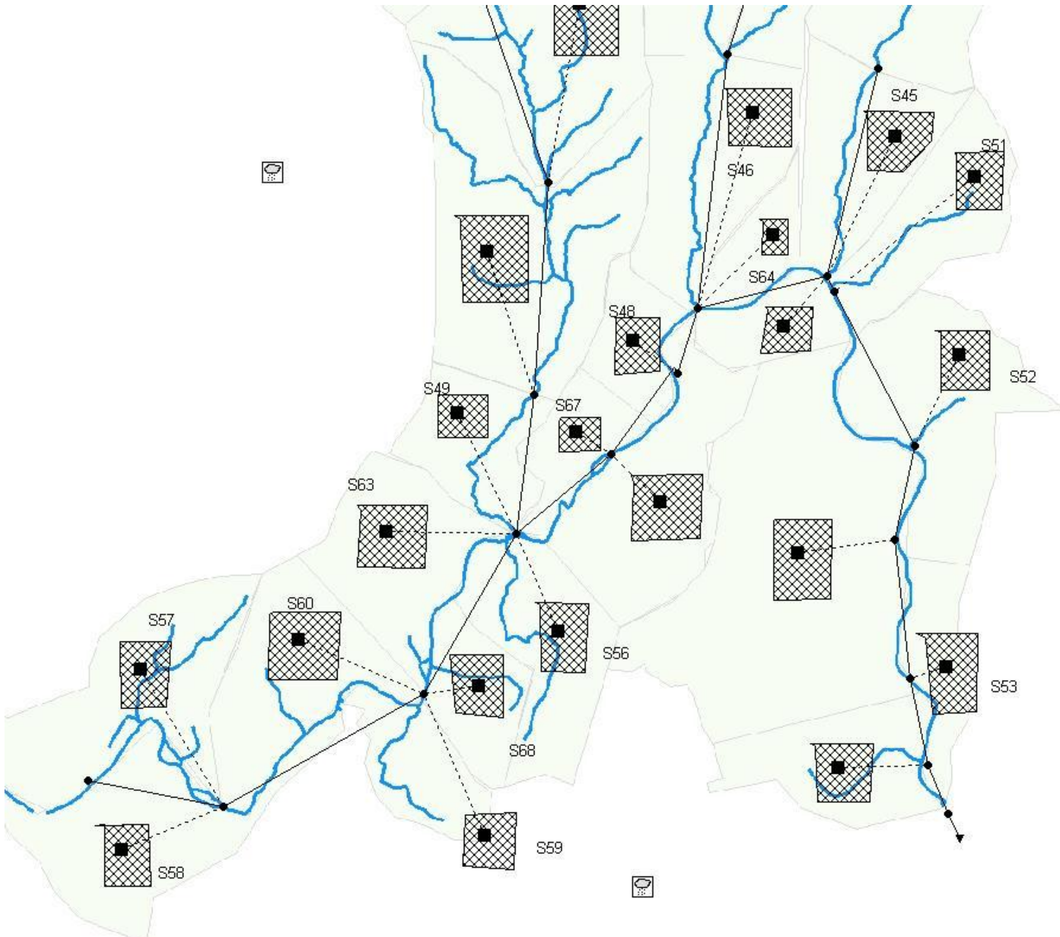

Figure III-2 – SWMM Model Inputs into the River

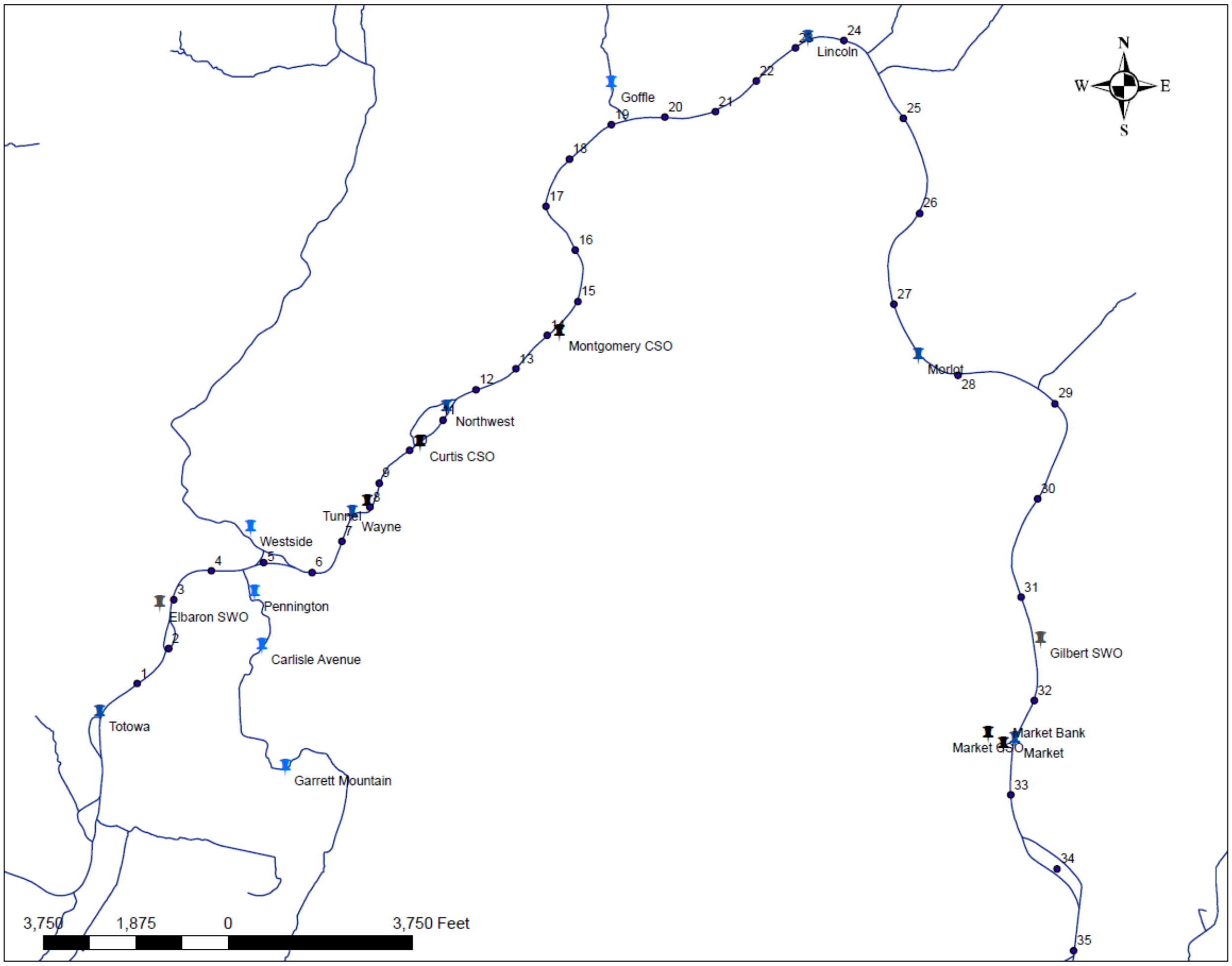

Figure III-3 – Node Positioning in WASP Segments

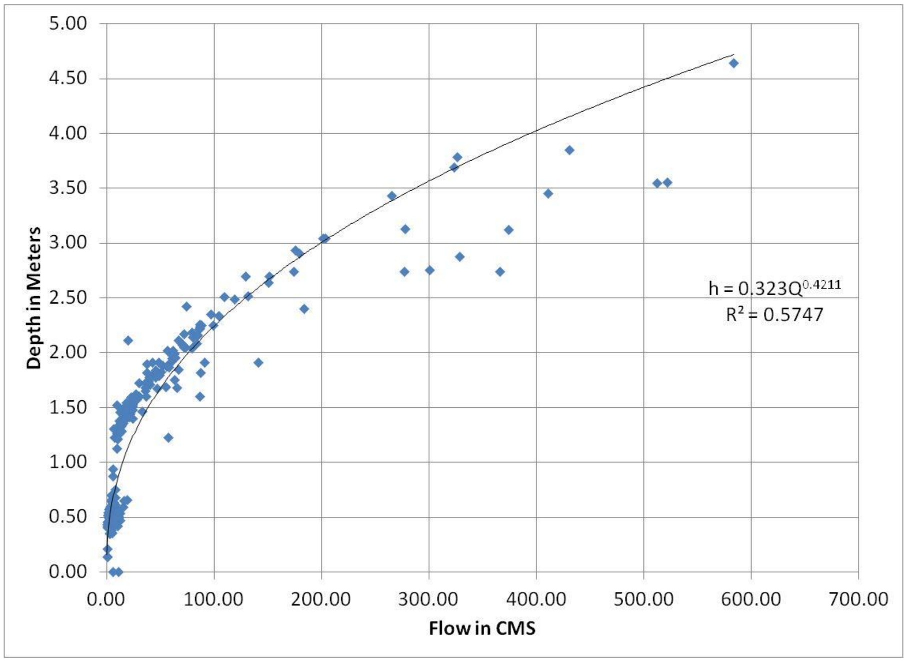

Figure III-4 – Depth Multiplier and Exponent and Little Falls

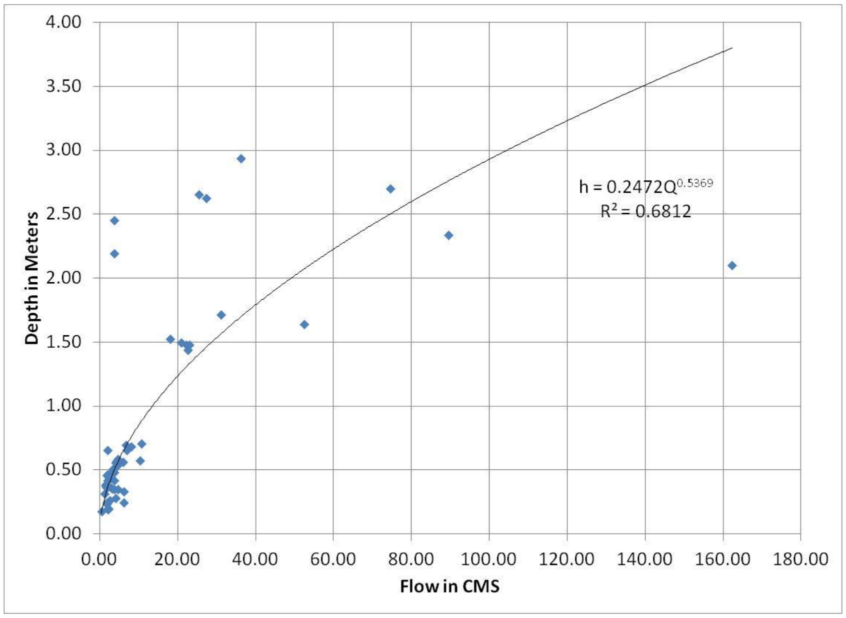

Figure III-5 – Depth and Velocity Multiplier at Great Falls

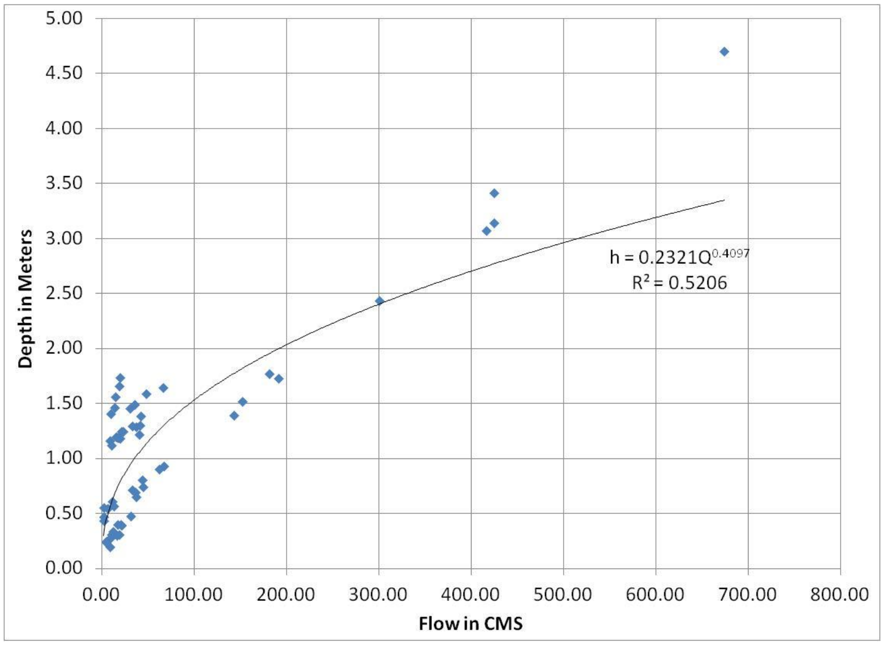

Figure III-6 – Depth Multiplier and Exponent at Dundee Dam

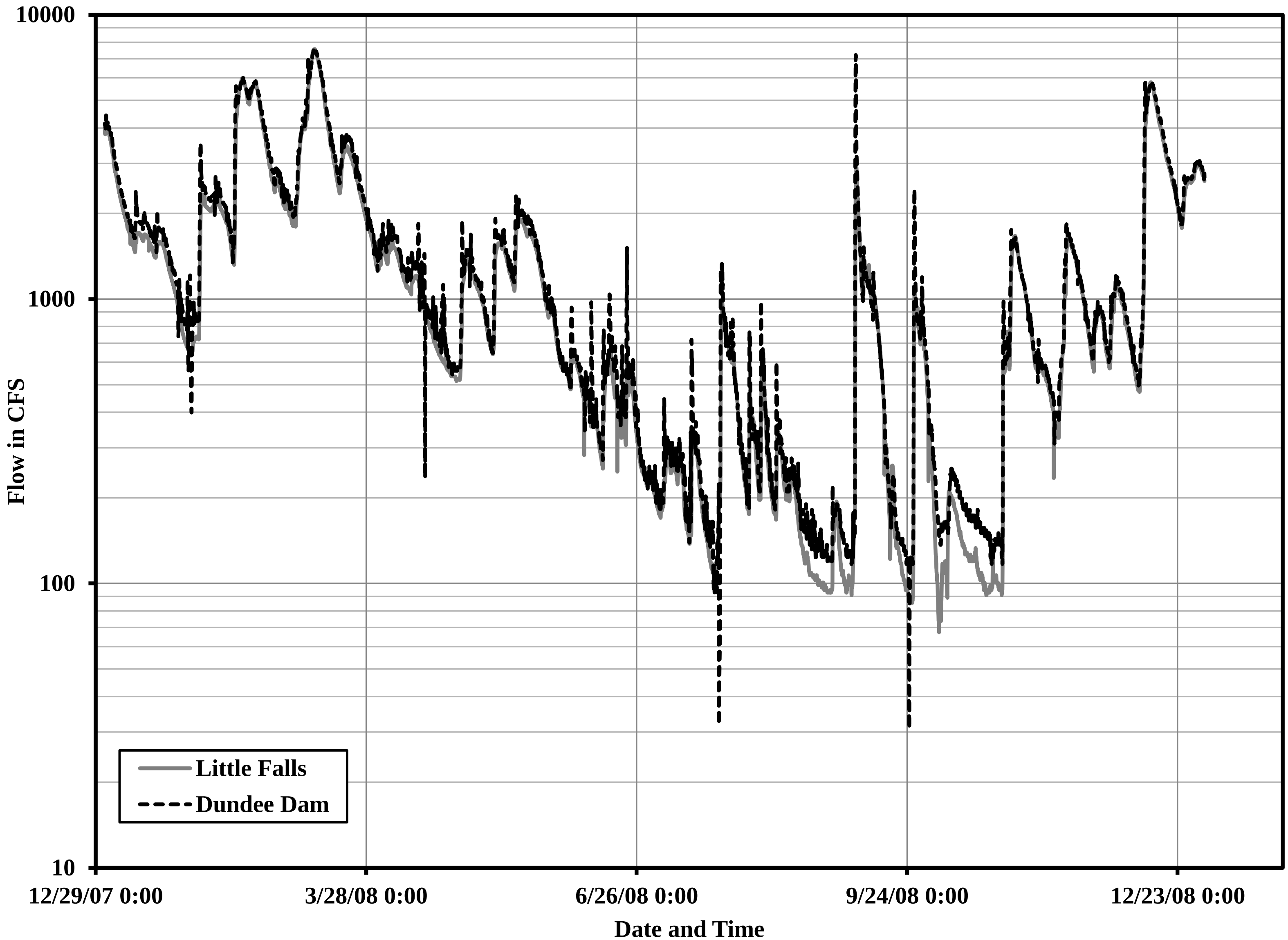

Figure III-7 – Little Falls and Dundee Dam Flow Data for Year 2008

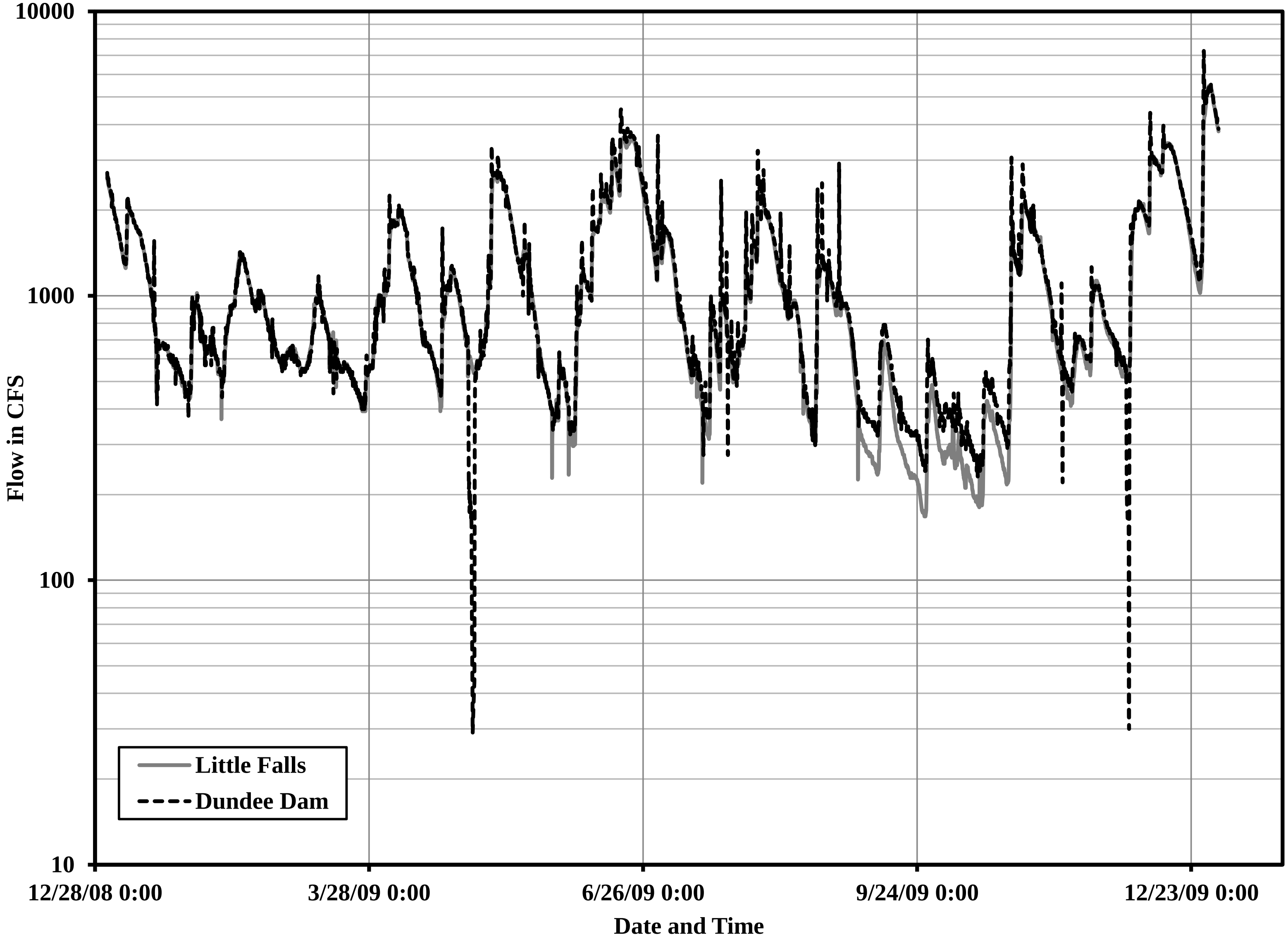

Figure III-8 – Little Falls and Dundee Dam Flow Data for Year 2009

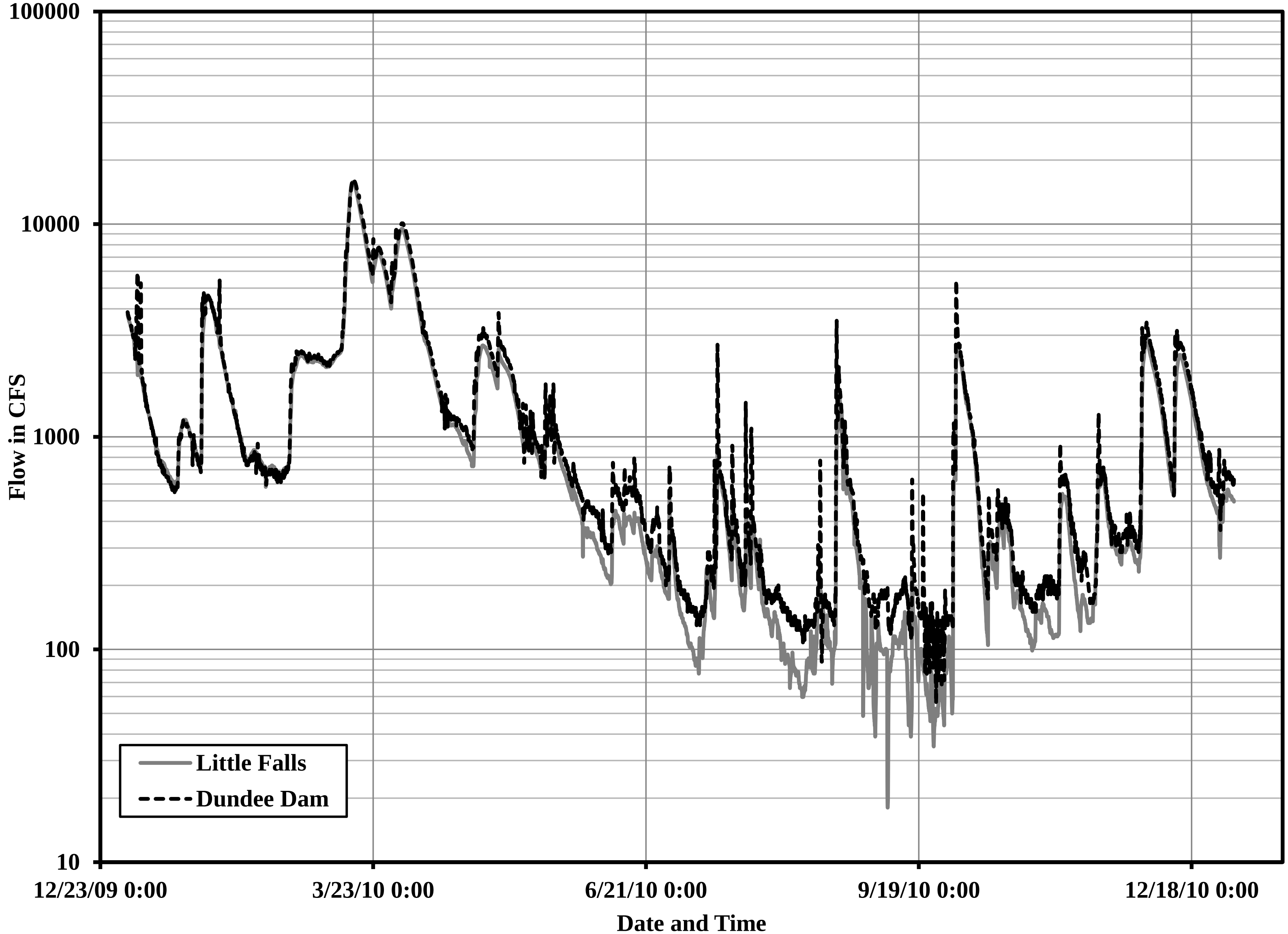

Figure III-9 – Little Falls and Dundee Dam Flow Data for Year 2010

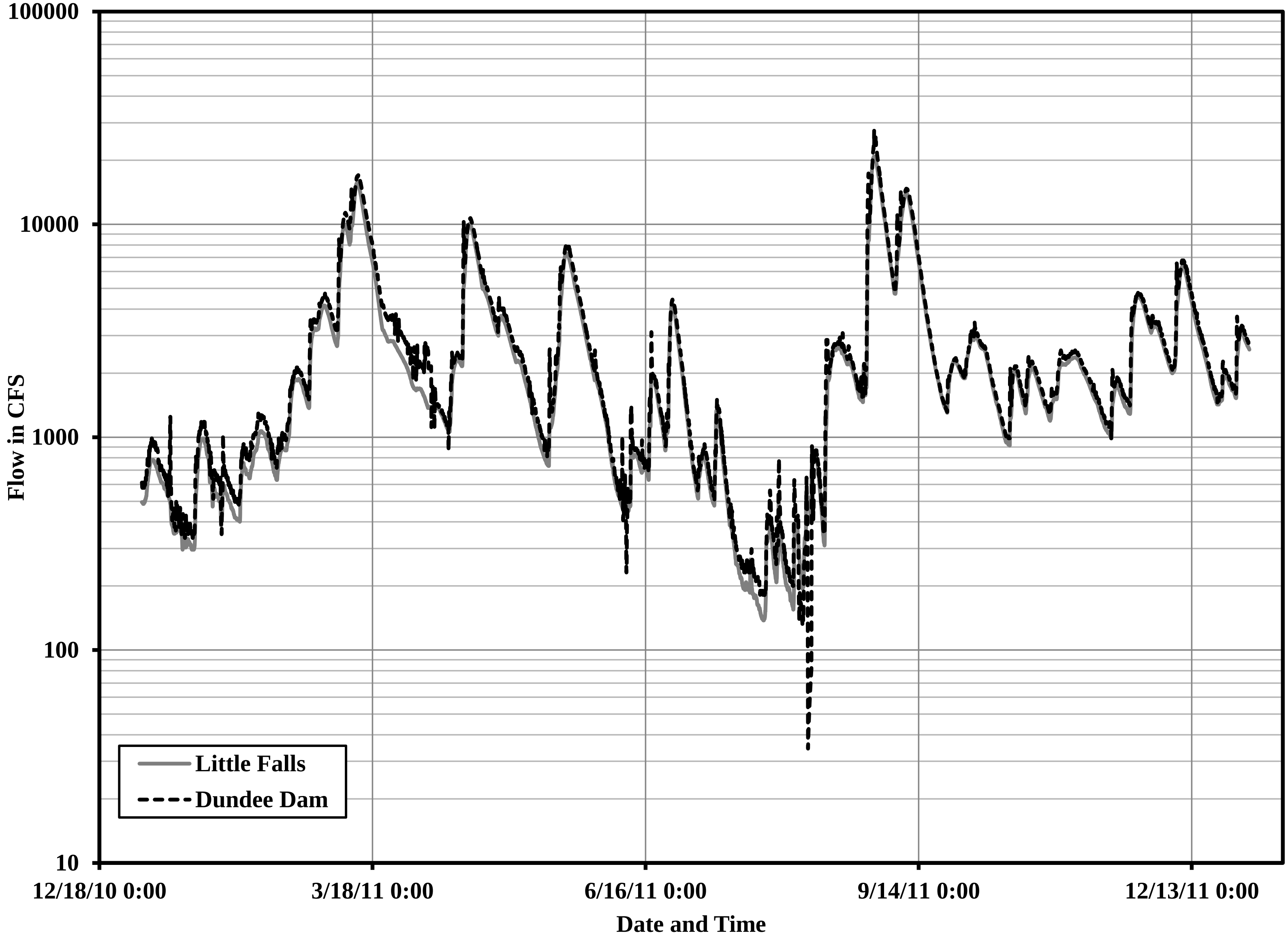

Figure III-10 – Little Falls and Dundee Dam Flow Data for Year 2011

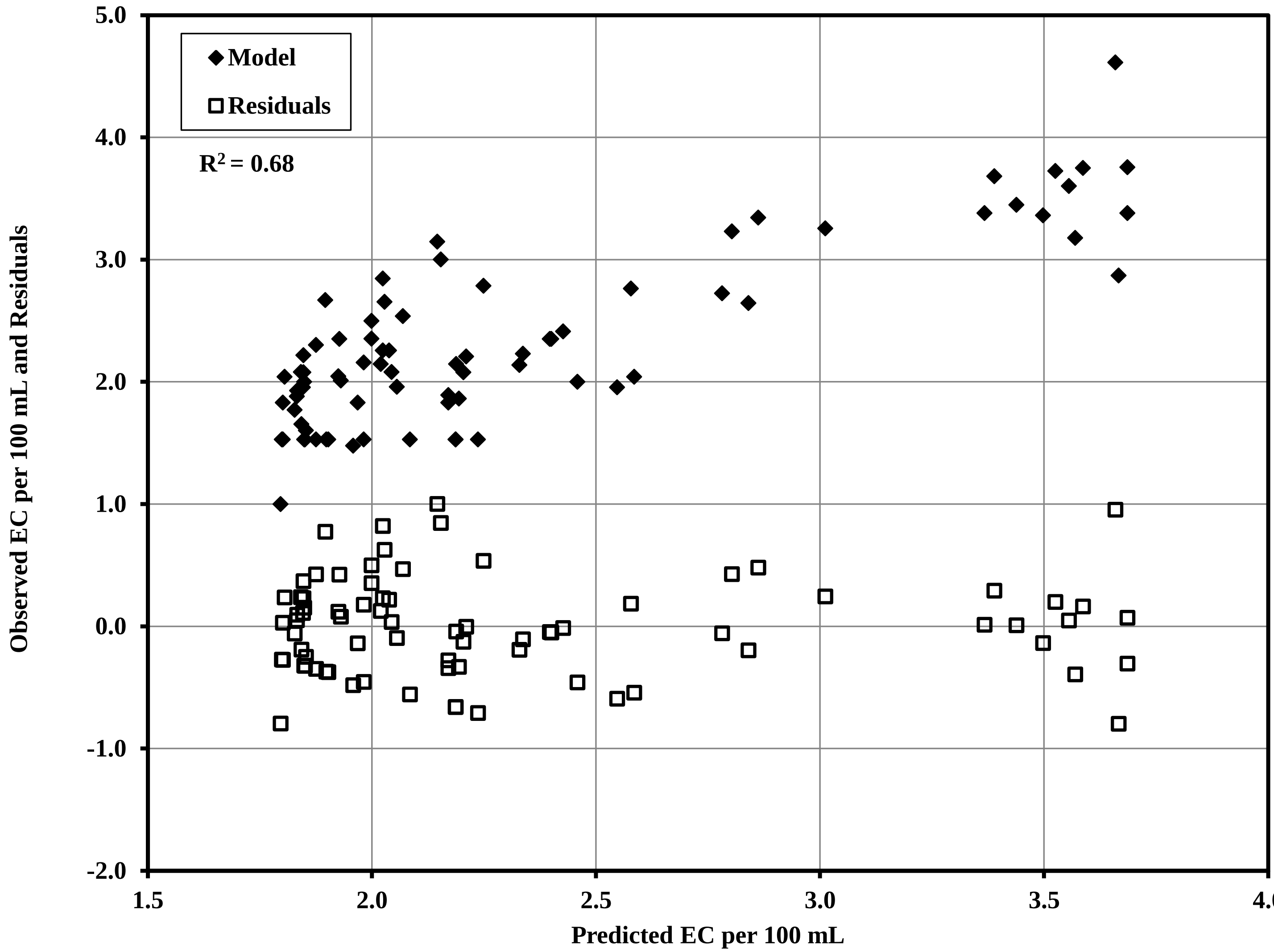

Figure III-11 – Predicted versus Observed results for Upstream Boundary

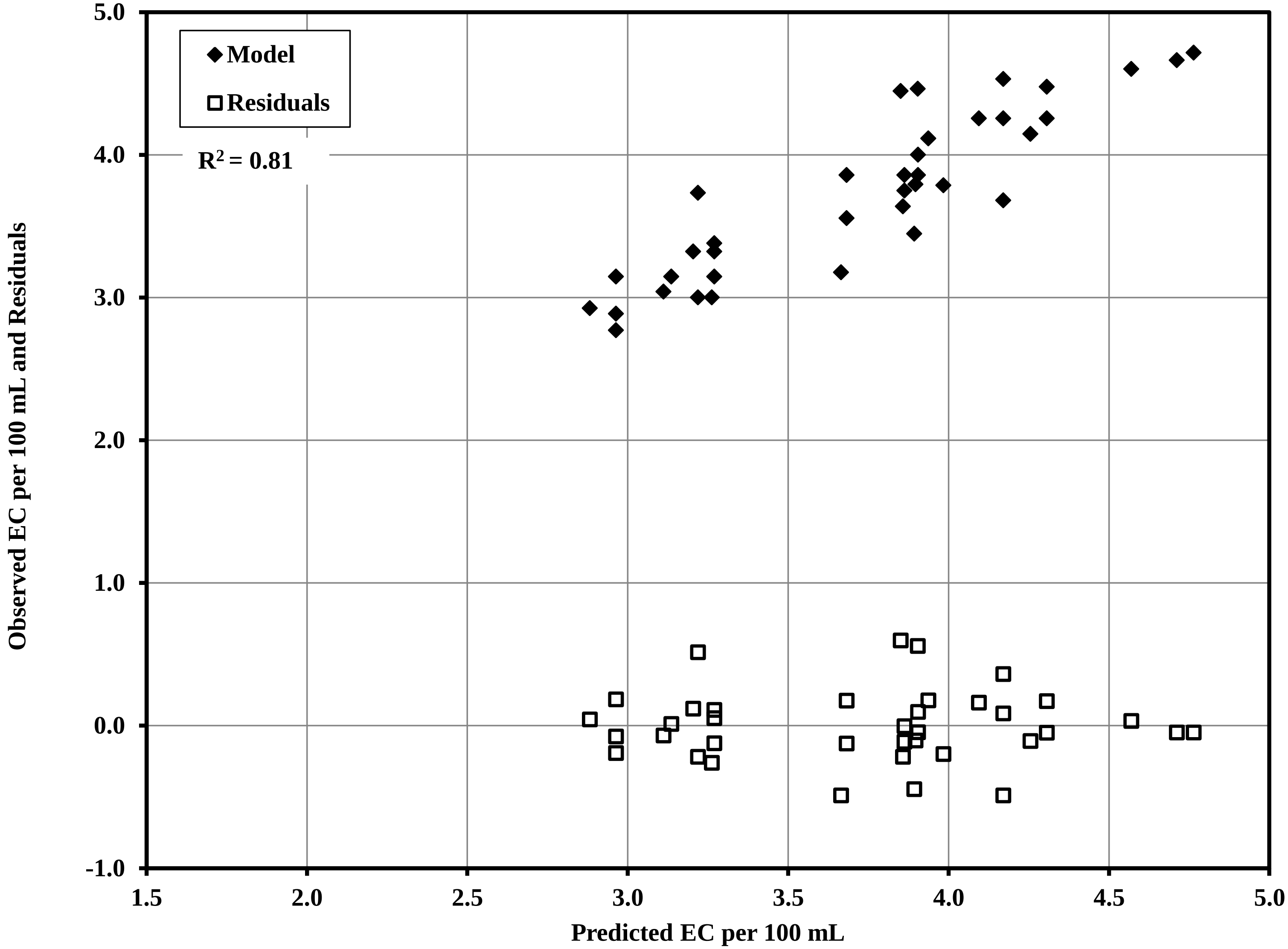

Figure III-12 – Predicted versus Observed results for Pennington Site

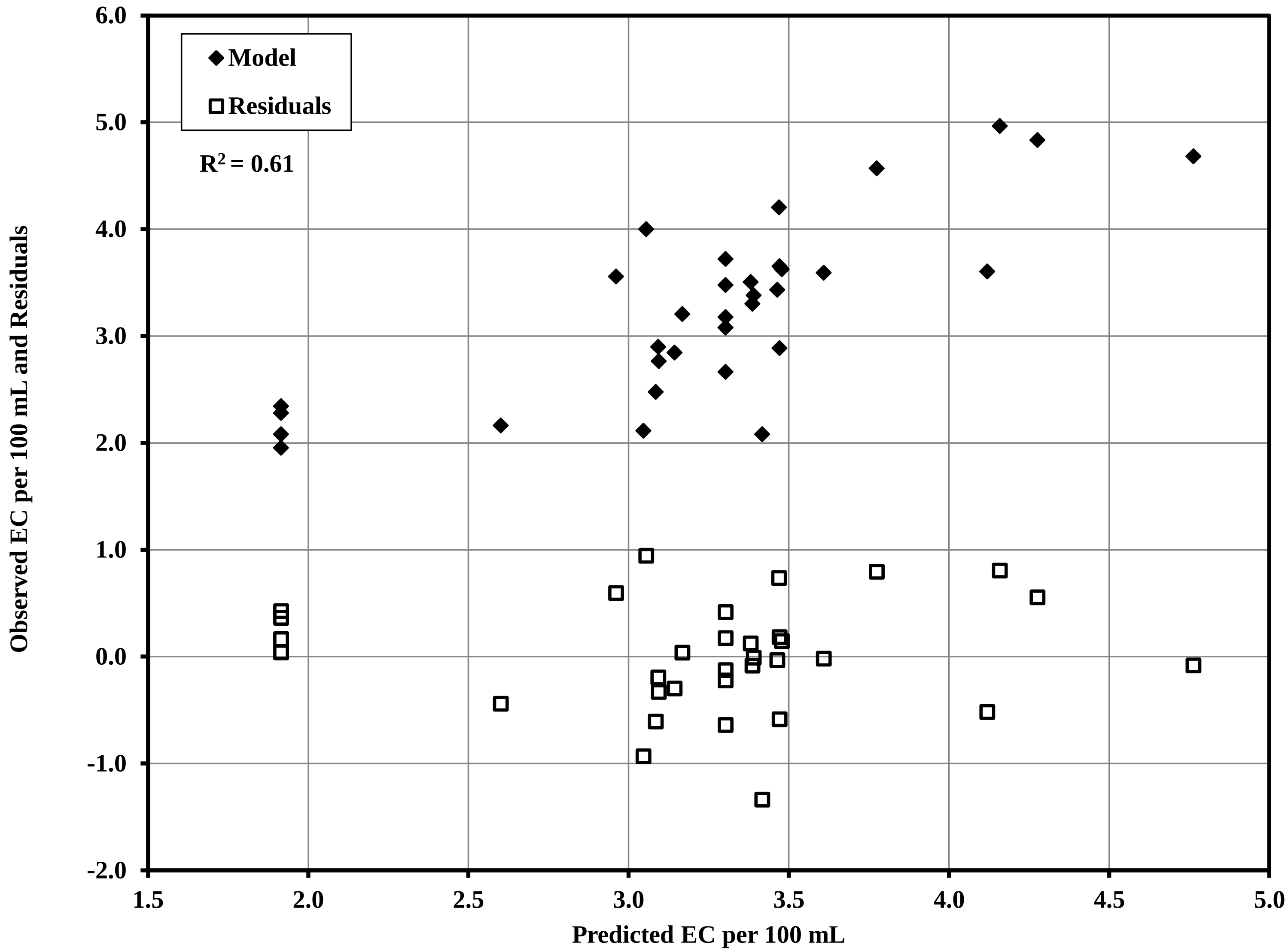

Figure III-13 – Predicted vs Observed for Westside Site

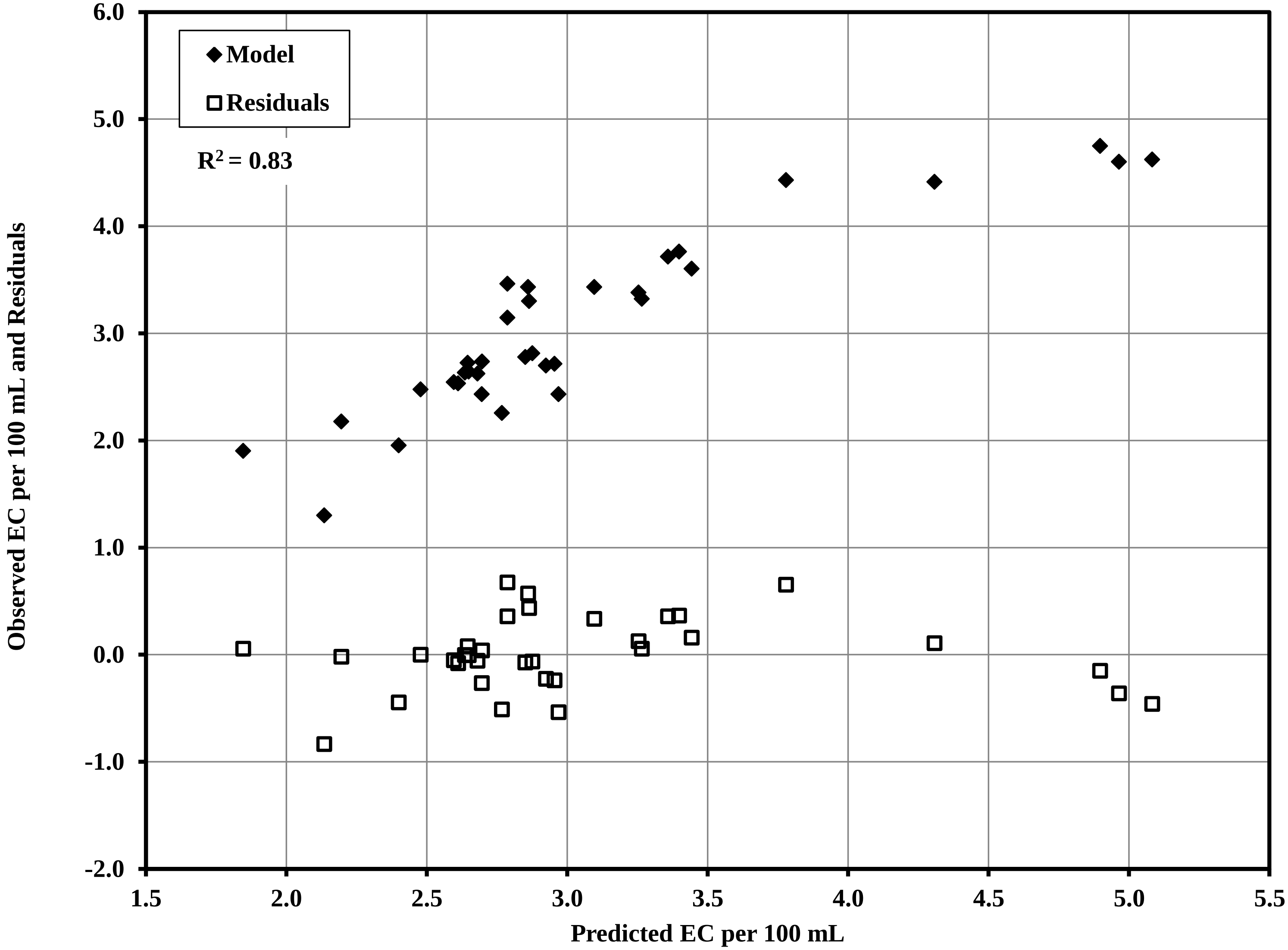

Figure III-14 –Predicted vs Observed for Goffle Site

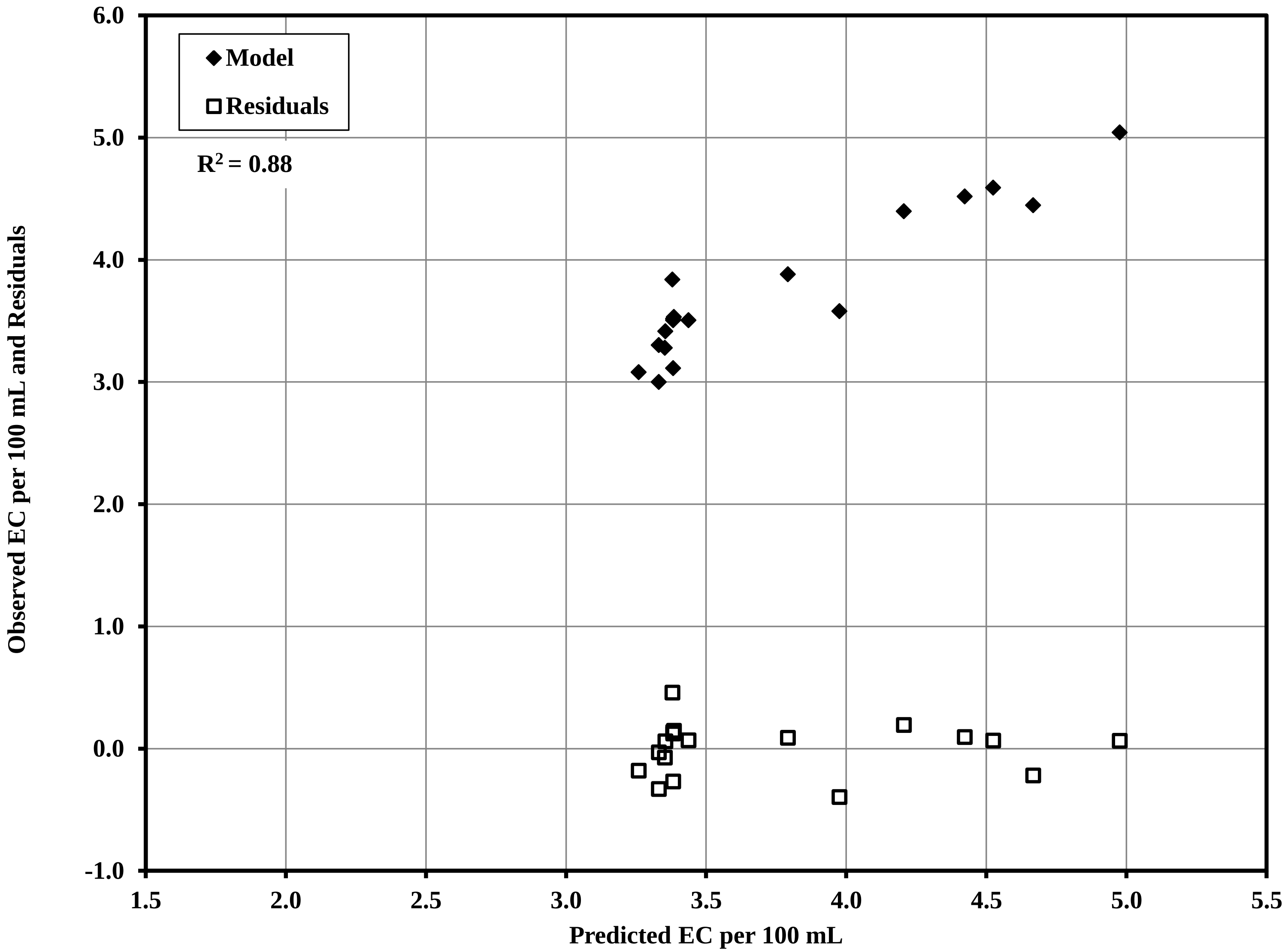

Figure III-15 – Modeled vs Observed for the Stormwater at Gilbert SWO

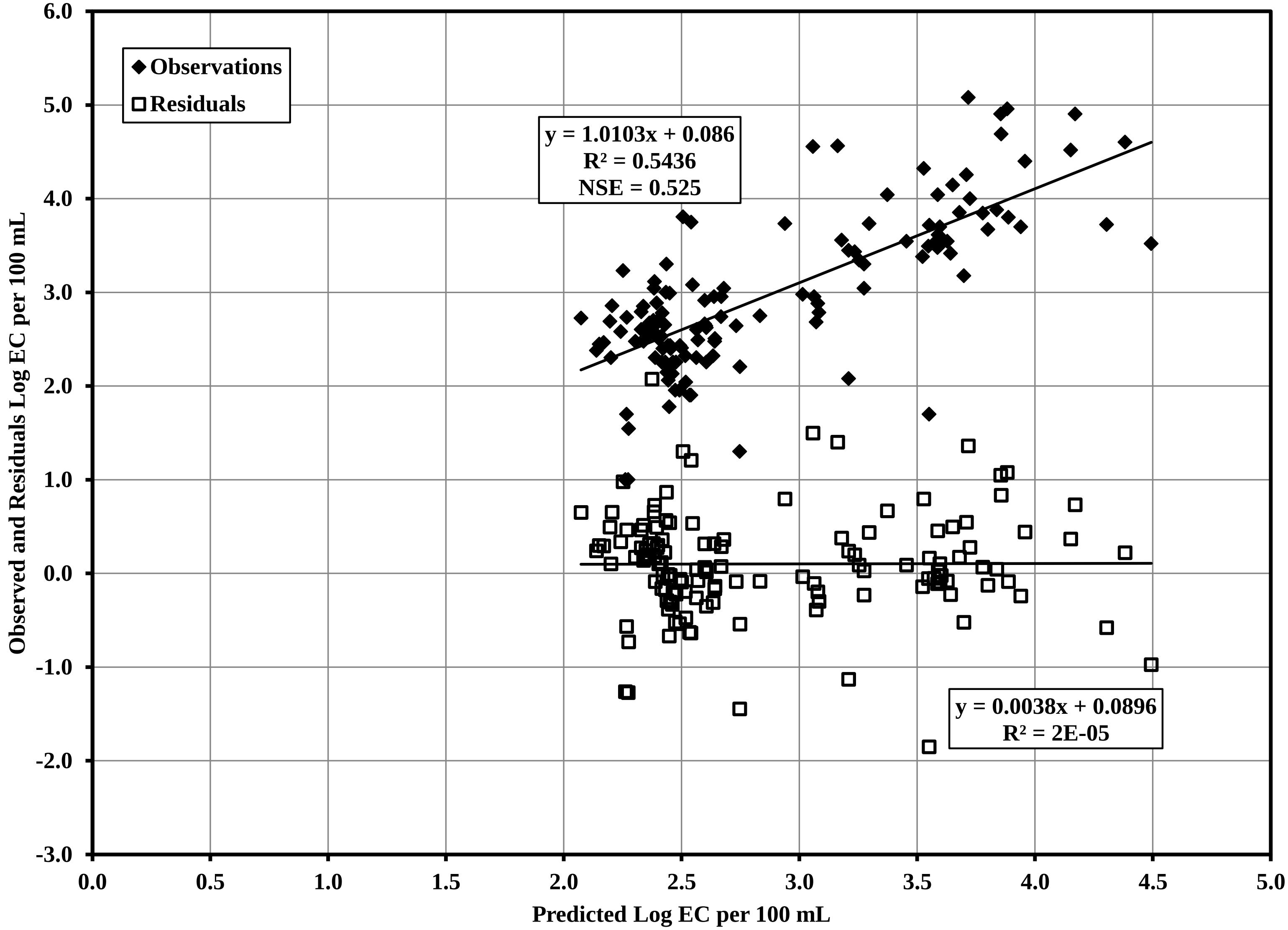

Figure III-16 – Calibration goodness-of-fit for all Sites (pooled)

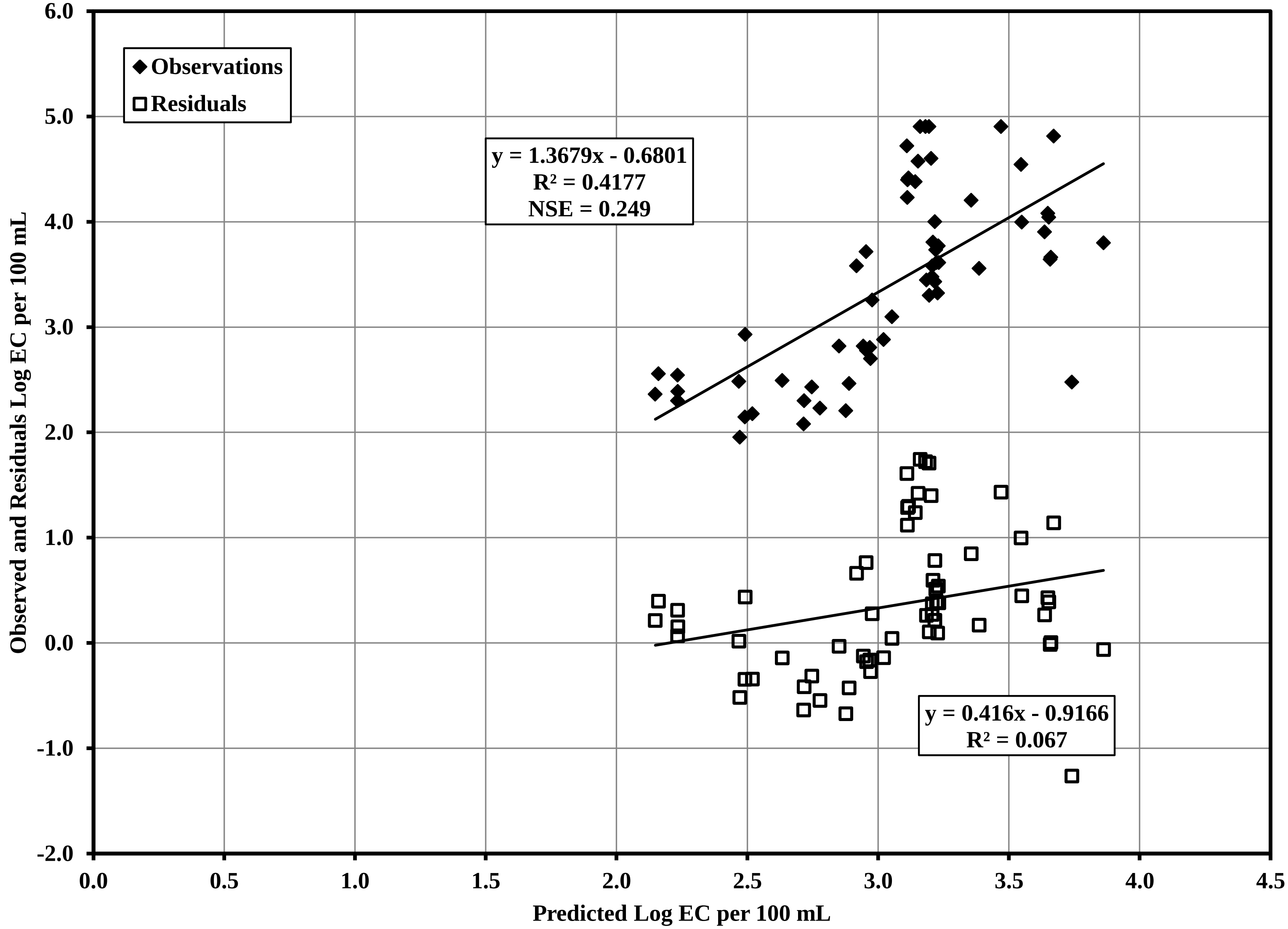

Figure III-17 – Validation goodness-of-fit for all Sites (pooled)

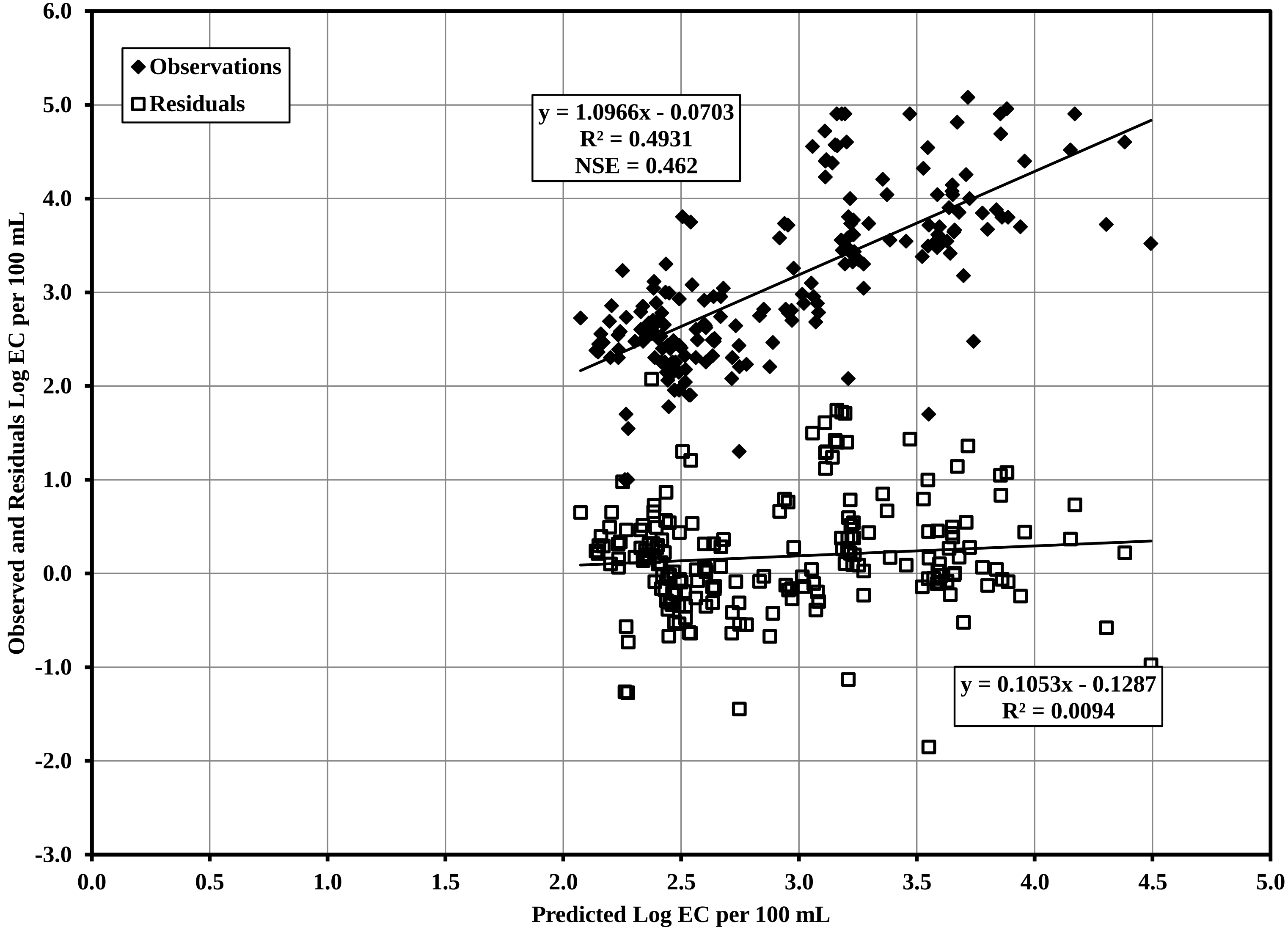

Figure III-18 – Goodness-of-fit for combination of calibration and validation data

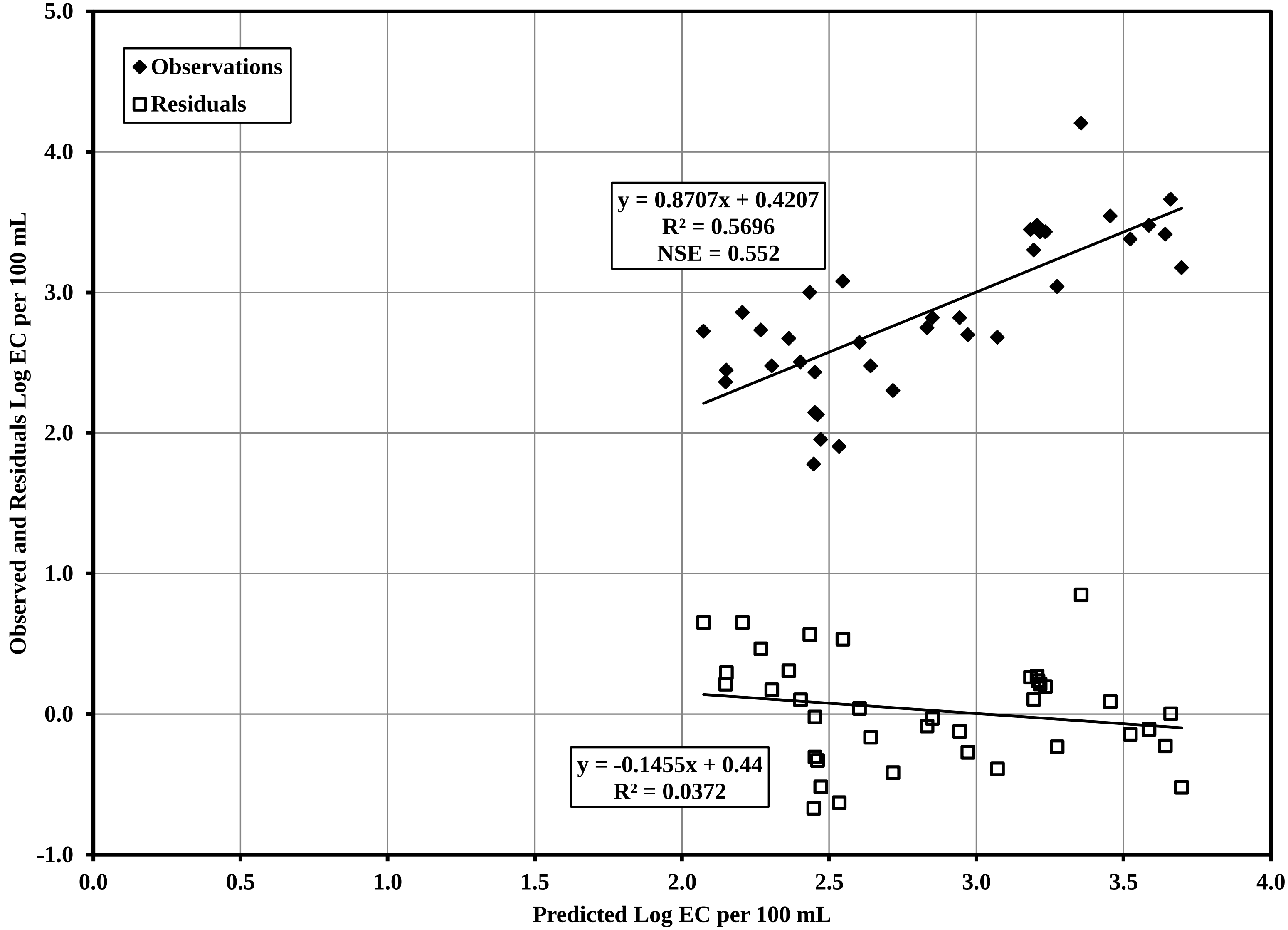

Figure III-19 – Goodness of Fit at Wayne

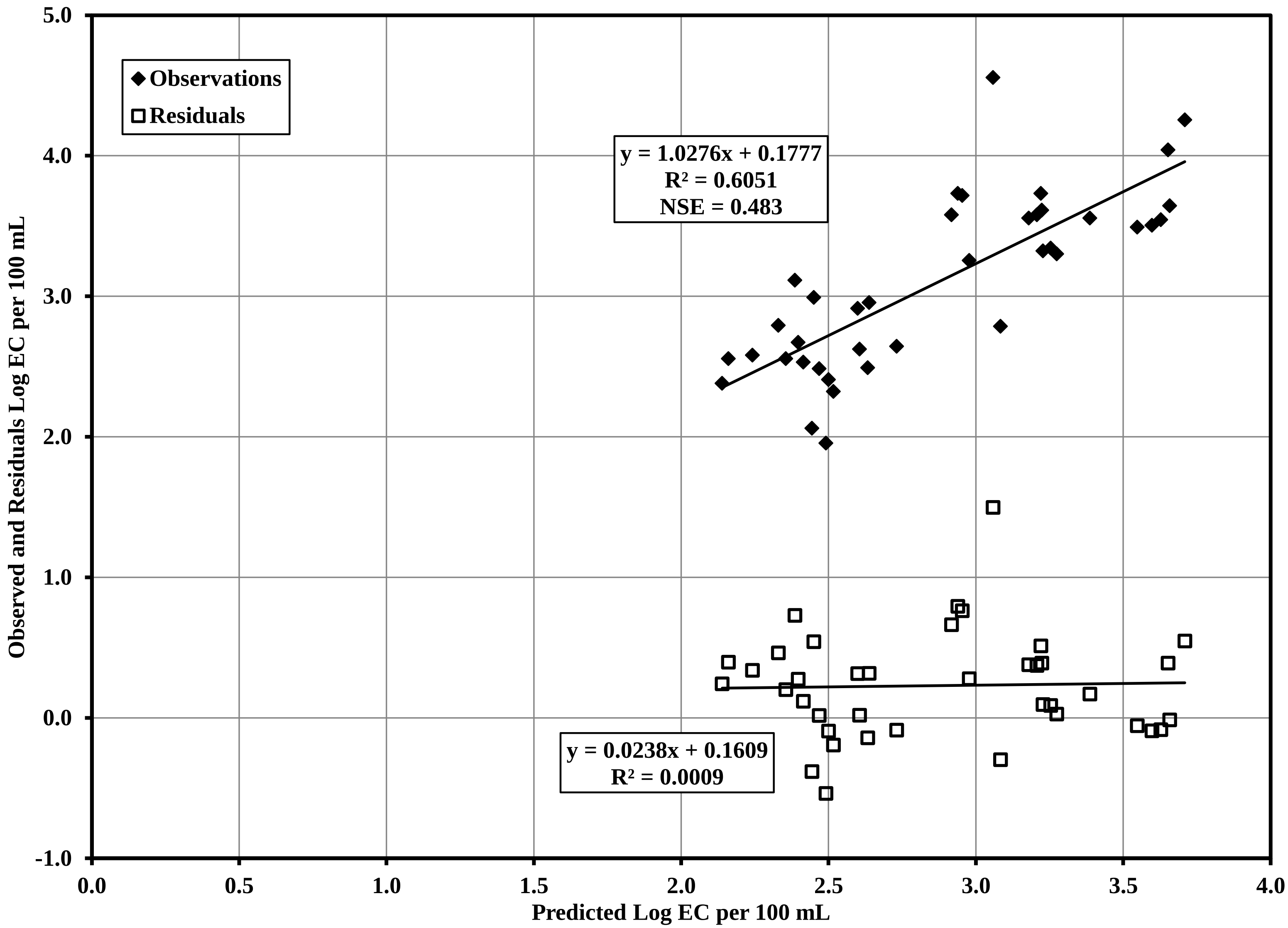

Figure III-20 – Goodness of Fit at Northwest

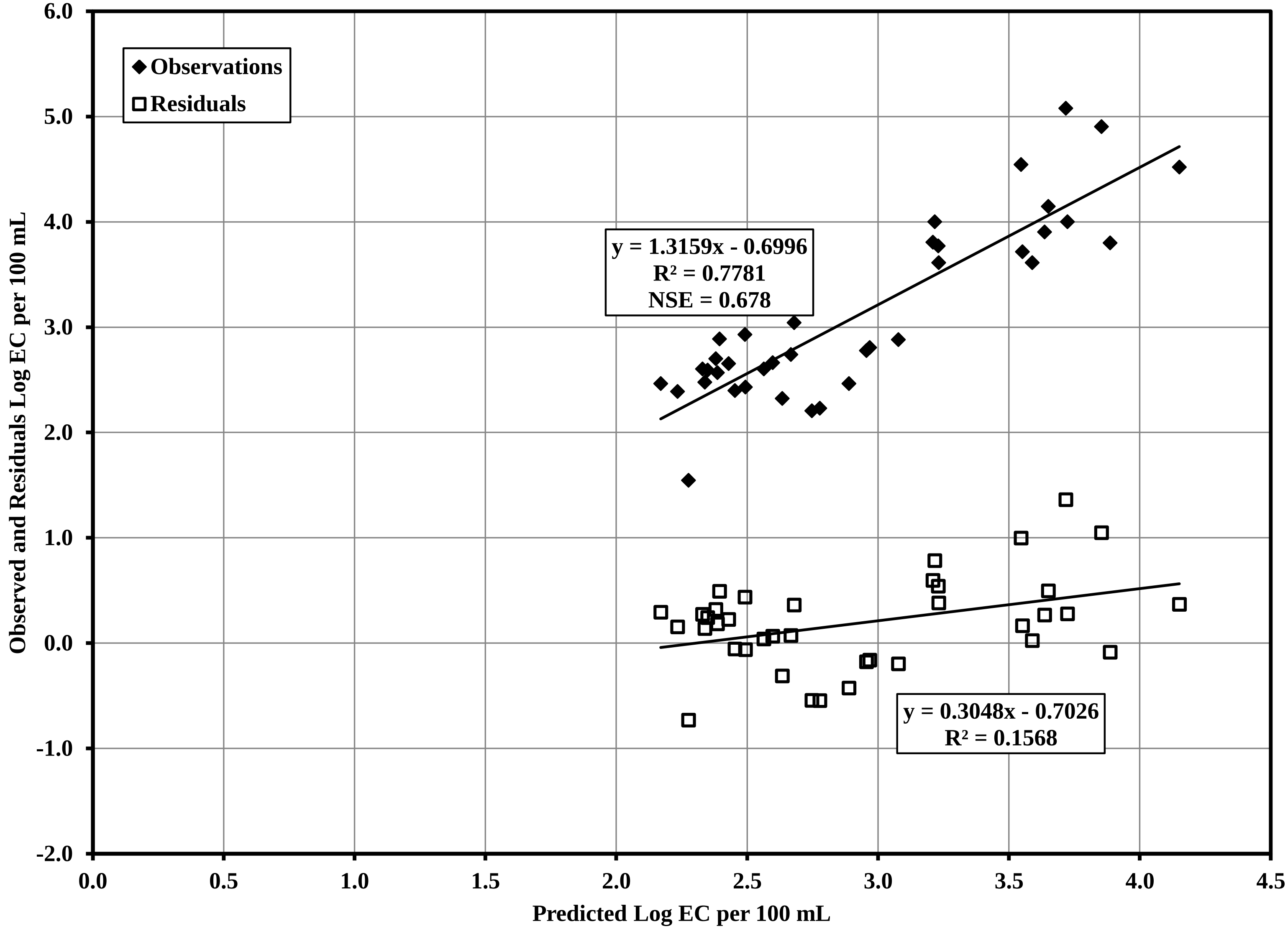

Figure III-21 – Goodness of Fit at Lincoln

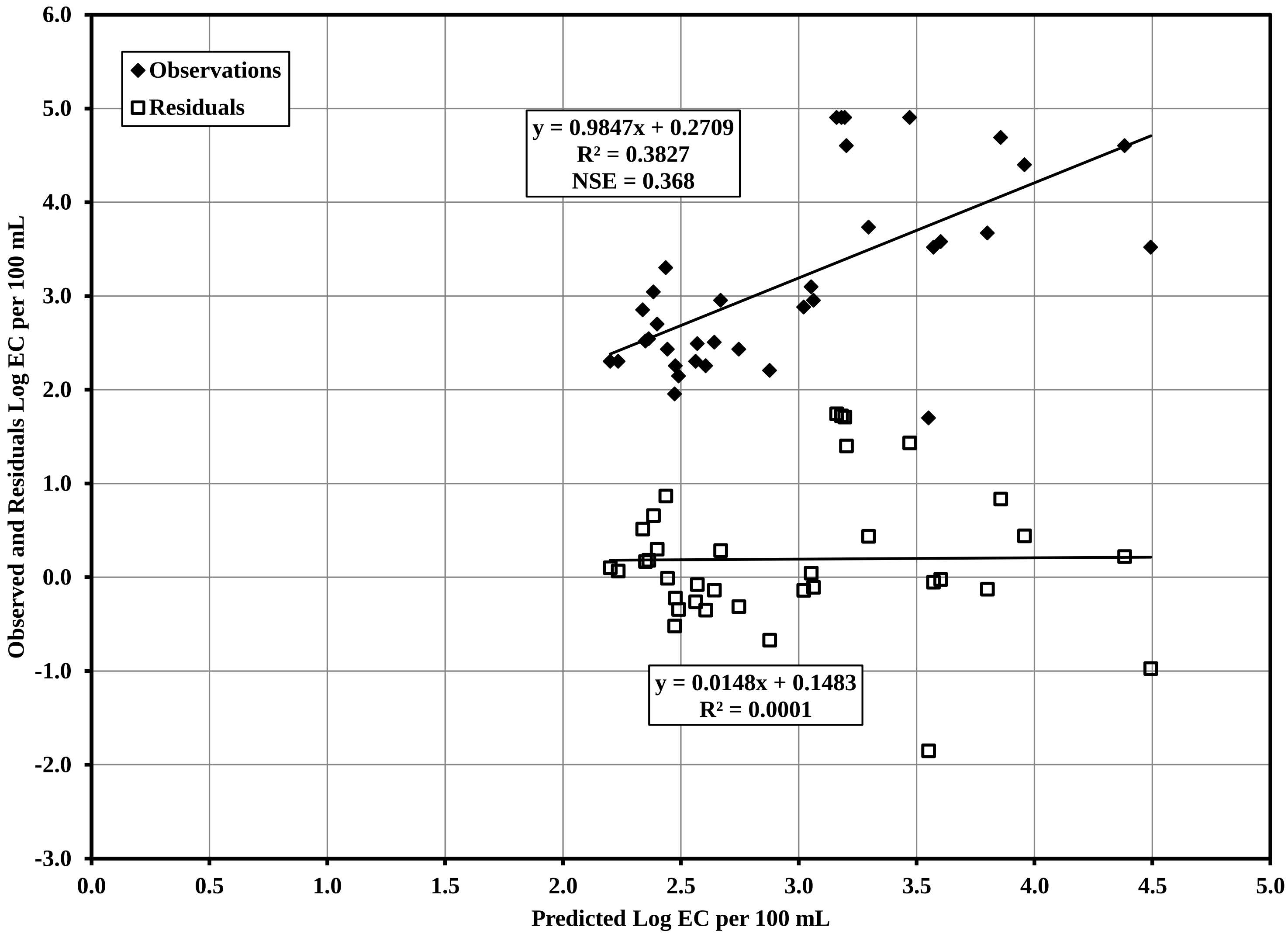

Figure III-22 – Goodness of Fit at Morlot

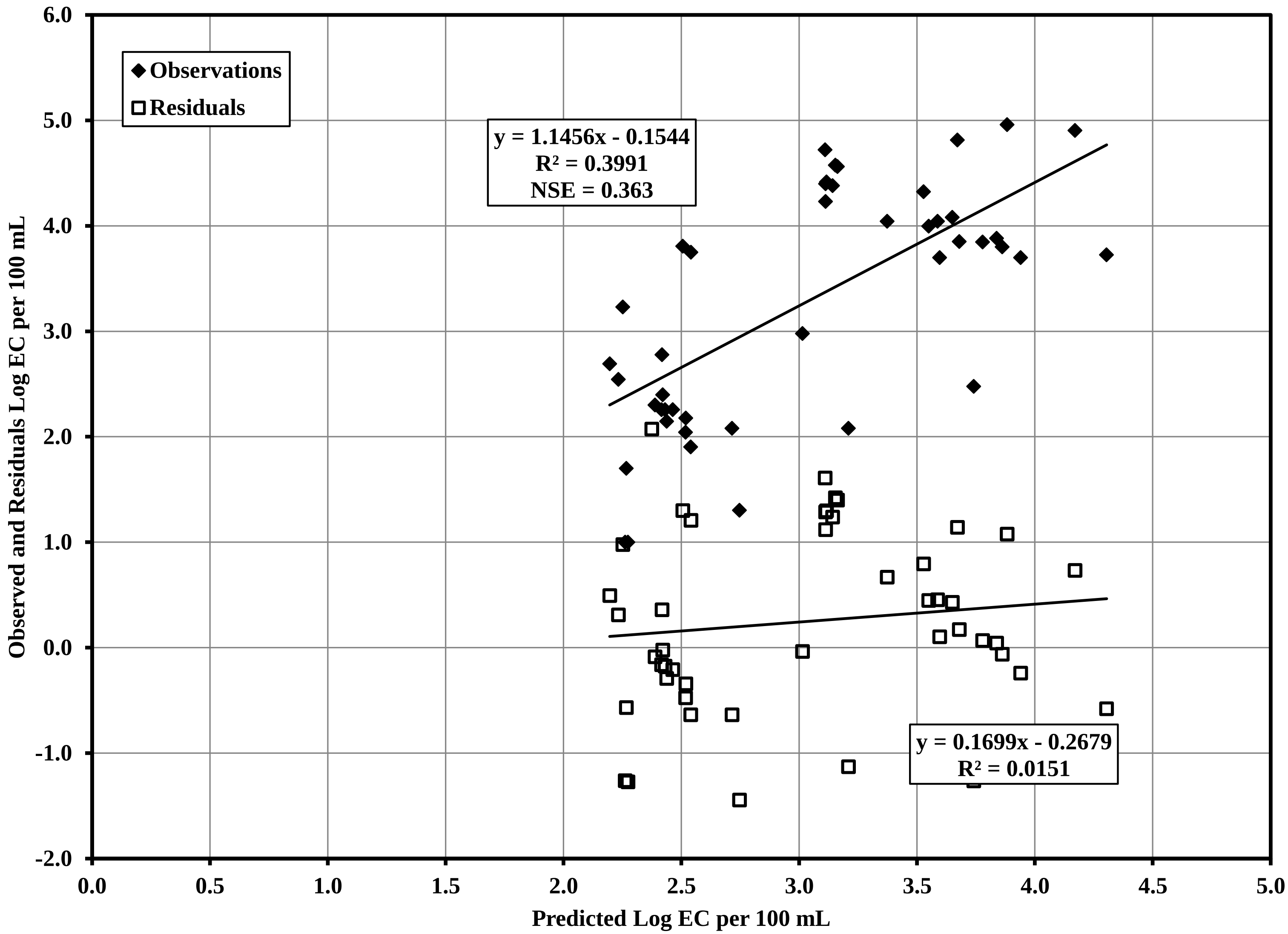

Figure III-23 – Goodness of Fit at Market Street

IV. Watershed Modeling Results

The calibrated water quality model was then used to simulate the existing conditions and the impacts of various sources for the years 2009, 2010 and 2011.

A. Existing Condition

The model predicted that the water quality standards would generally be violated over the stretch of the river for all three years in which the model was simulated. The 30-day moving geometric mean (GM) in 2009 was never below 300 EC per 100 mL (Figure IV-1). The 30-day GM was well above for the water quality target of 126 EC per 100 mL for the entire period. The spatially averaged annual geometric mean for 2009 was 772 EC per 100 mL. The seasonal GM (May 15 to September 15) was 663 EC per 100 mL. The spatial average was calculated by finding the geometric mean of concentrations in all segments. The downstream boundary GM is higher with an annual value of 1084 EC per 100 mL and a seasonal value of 1373 EC per 100 mL. The concentrations increased (Figure IV-2) going downstream due to additional sources such as combined sewer outfalls, storm water, and tributaries.

There were numerical instabilities in the WASP runs of 2010 and 2011. These were especially observed during extremely high flow conditions (March 2010 and September 2011). Therefore, simulations from these periods were removed as they are unreliable.

In the year 2010 the 30-day GM also never went below 300 EC per 100 mL (Figure IV-3). The year 2011 had relatively better water quality than years 2009 and 2010, with a few excursions below 300 EC per 100 mL (Figure IV-5). The concentrations tended to increase, as expected, towards the downstream end of the reach (Figure IV-4 and Figure IV-6). The spatially averaged annual GM in 2010 was 724 EC per 100 mL and the seasonal GM was 708 EC per 100 mL. The corresponding numbers for 2011 were 478 EC per 100 mL and 608 EC per 100 mL. 2010 is the only year in which the annual GM is greater than the seasonal GM.

Over the three year period, the warm (May 15 to September 15) season GM was predicted to be higher than the overall GM. The plot shows a significant jump in the segment where the Stony Brook and Molly Ann enter the river, and another jump at the Morlot 2 segment. Morlot 2 is the segment where Figure III-3 shows that an unnamed tributary enters the river. The SWMM model simulation showed that the total storm water flow entering this segment is third highest following the segment where Stony Brook and Molly Ann enter and the segment where the Goffle Brook enters.

B. Source Assessment

a. Effect of CSOs

The removal of CSO loading seems to have very little effect (Figure IV-7, Figure IV-9, and Figure IV-11). The predicted annual GM reduced from 663 to 627 in 2009, 724 to 672 in 2010, and 478 to 461 in 2011 representing a 5%, 7%, and 3% drop respectively. The corresponding reductions for the seasonal GM were 8%, 10%, and 8% respectively for years 2009, 2010, and 2011.

There is a relatively larger impact at the downstream boundary of the stretch (Figure IV-8Figure IV-10Figure IV-12). The annual drops were 19%, 23%, and 8% and the seasonal drops were 25%, 32%, and 16% respectively for years 2009, 2010, and 2011. Therefore, eliminating the CSOs has a significant impact on the downstream boundary water quality.

b. Effect of SWOs

The predicted water quality was more sensitive to stormwater outfall (SWO) discharges than to CSOs (Figure IV-13, **Figure IV-15**, and **Figure IV-17**). For the purpose of this analysis, all tributaries are included as part of the stormwater since beyond the base flow, tributaries carry stormwater. The stormwater concentration was reduced by 90% to assess the impact of stormwater as a pollutant source.

This change decreased the simulated annual GM by 26%, 36%, and 27% for years 2009, 2010, and 2011, respectively. The corresponding figures for seasonal GM were 26%, 55%, and 32%. Spatially, the stretch became more homogenized with the reduction of stormwater loads (Figure IV-14, **Figure IV-16**, and **Figure IV-18**). The downstream boundary water quality is improved by 30%, 44%, and 42% annually, and 30%, 58%, and 41% seasonally for years 2009, 2010, and 2011 respectively.

c. Effect of both CSOs and SWOs

The effect of both eliminating the CSOs and reducing stormwater concentrations by 90% is, of course, more significant (**Figure IV-19**, **Figure IV-20**, **Figure IV-21**, **Figure IV-22**, **Figure IV-23**, and **Figure IV-24**). The annual GM drops were 35%, 50%, and 31%, the seasonal GM drops were 32%, 61%, and 38%, the downstream end annual drops were 50%, 67%, and 49%, and the downstream end seasonal drops were 51%, 76%, and 55%.

The results of the effects of sources are summarized in Table IV-1.

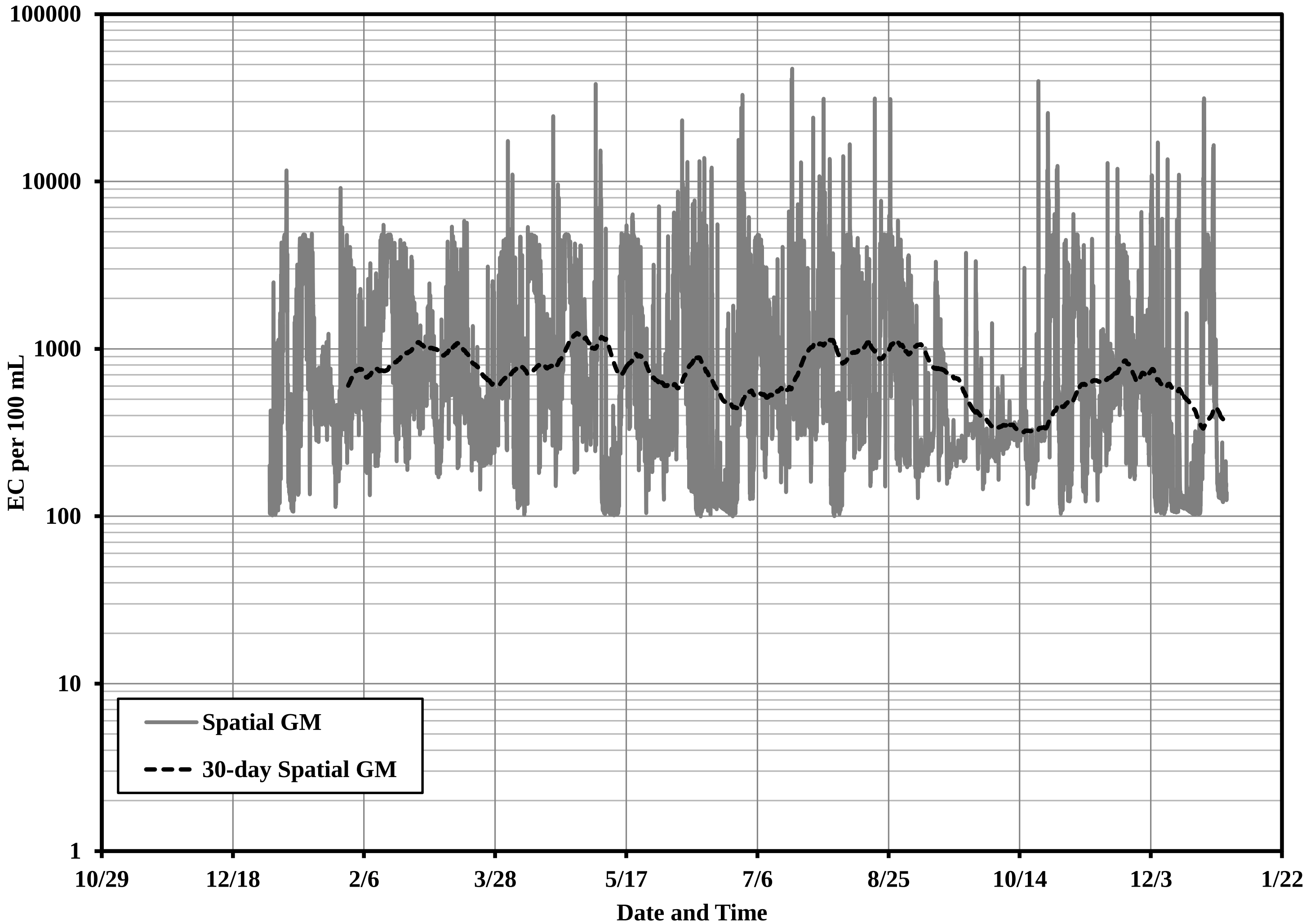

Figure IV-1 – Spatially Averaged Simulated Geometric Mean (2009)

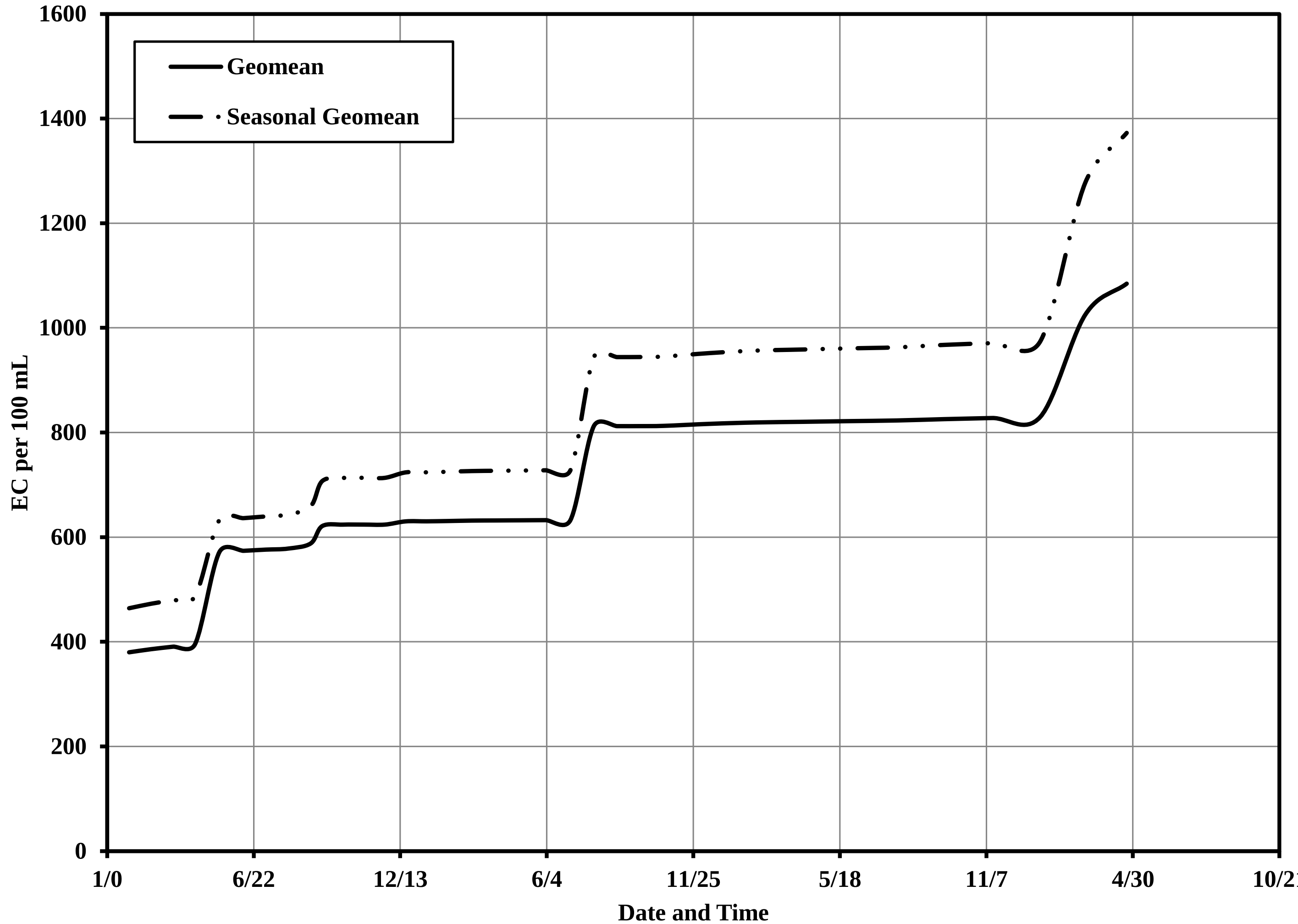

Figure IV-2 – Predicted Annual and Seasonal GM by Segments (2009)

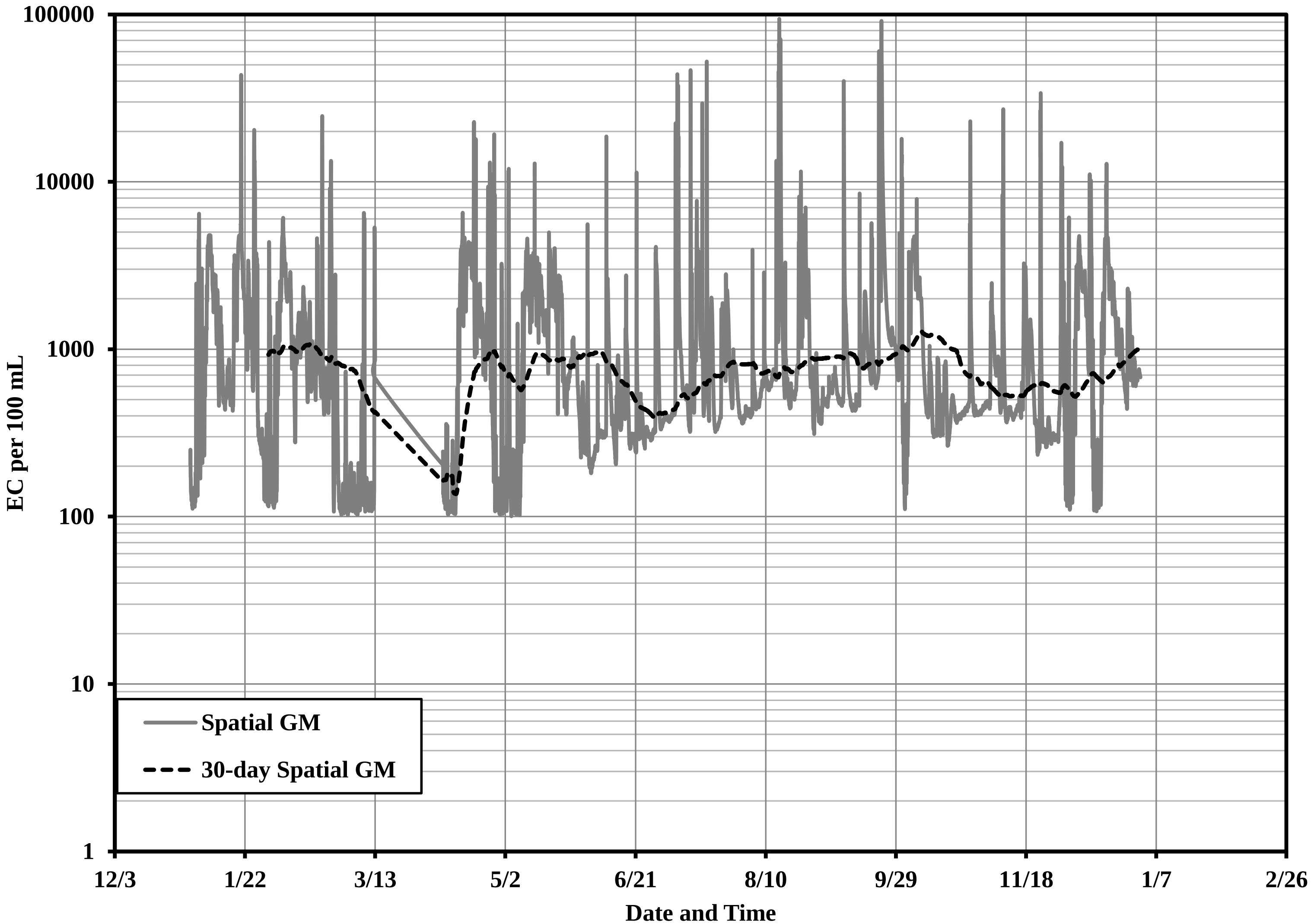

Figure IV-3 – Spatially Averaged Simulated Geometric Mean (2010)

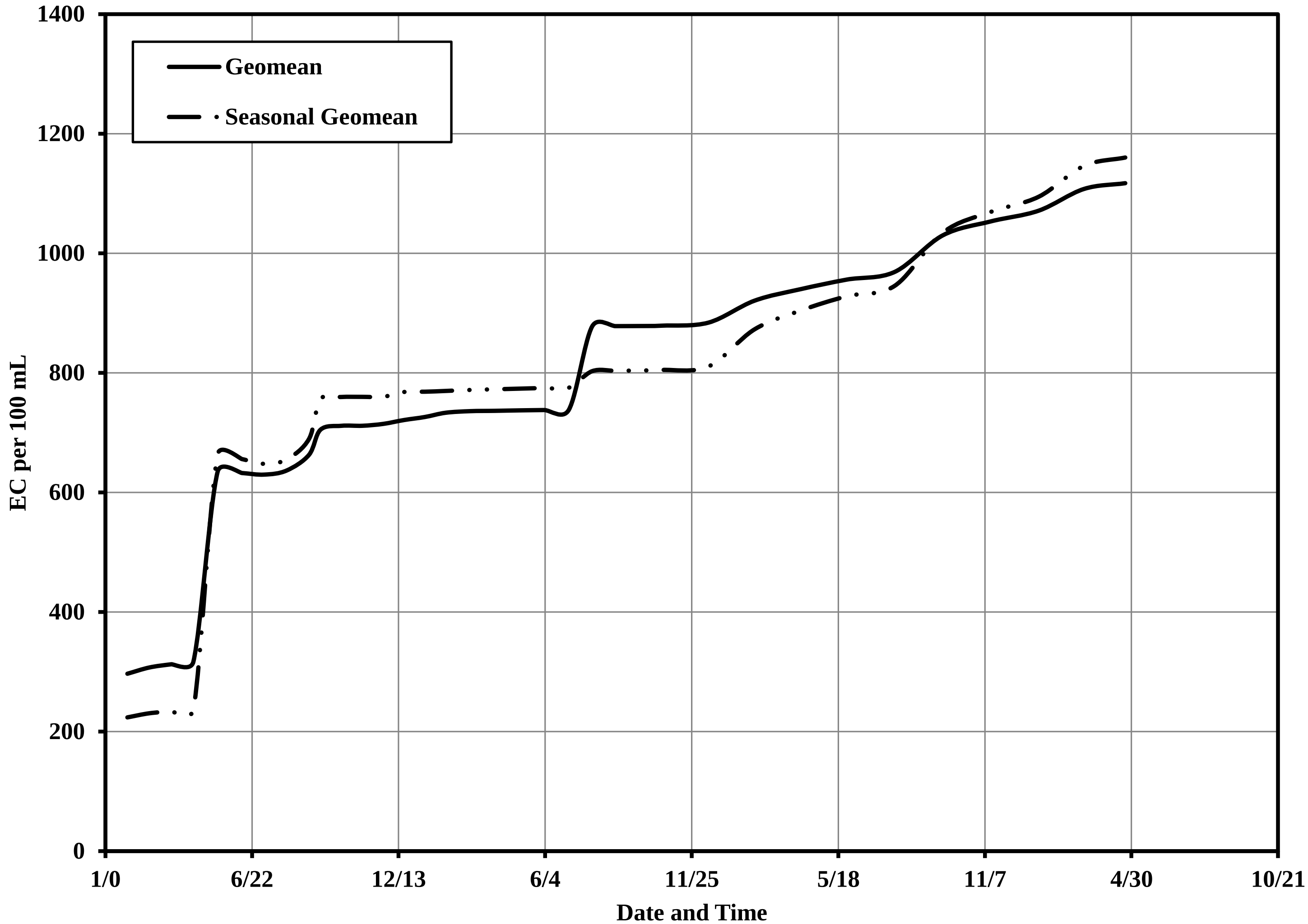

Figure IV-4 – Predicted Annual and Seasonal GM by Segments (2010)

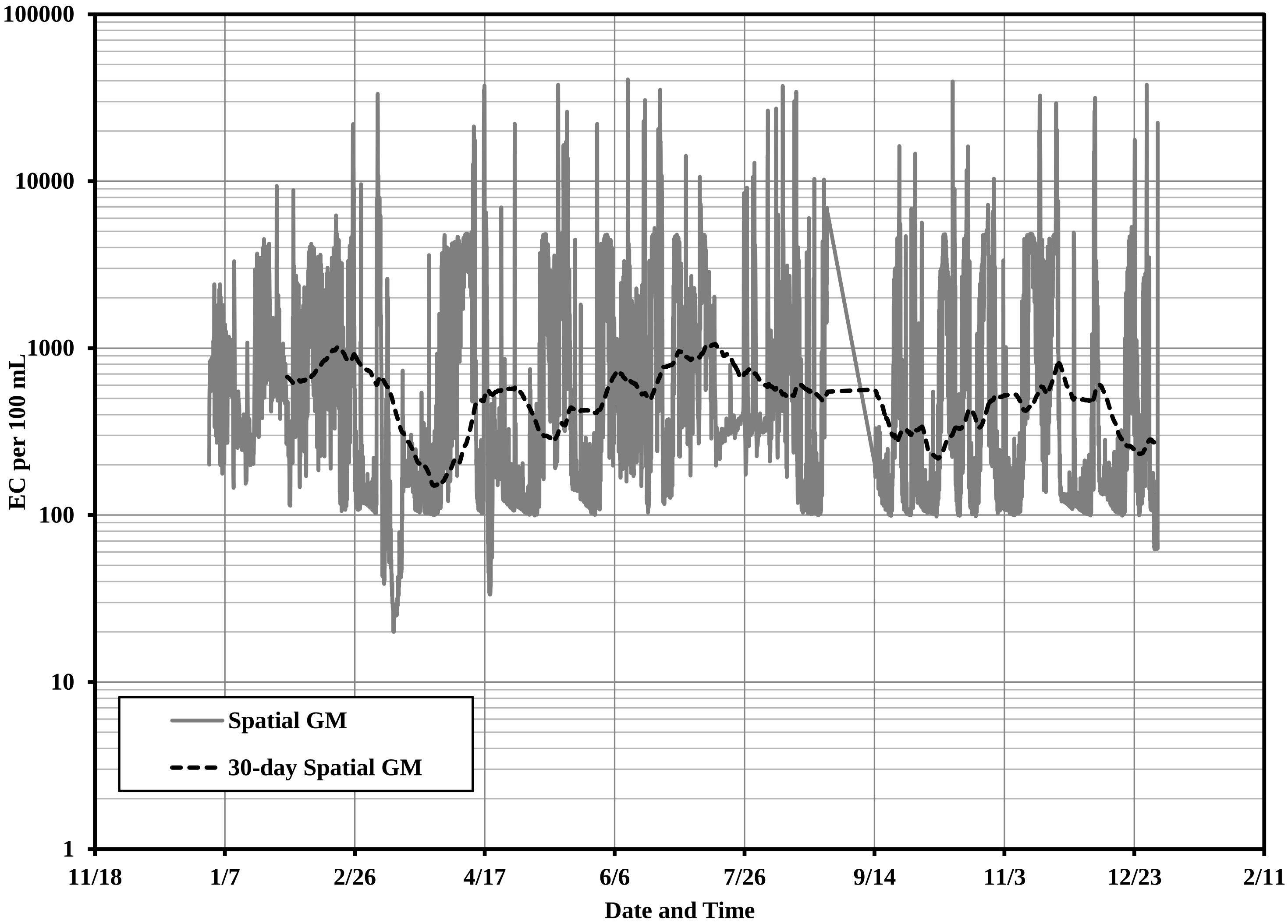

Figure IV-5 – Spatially Averaged Simulated Geometric Mean (2011)

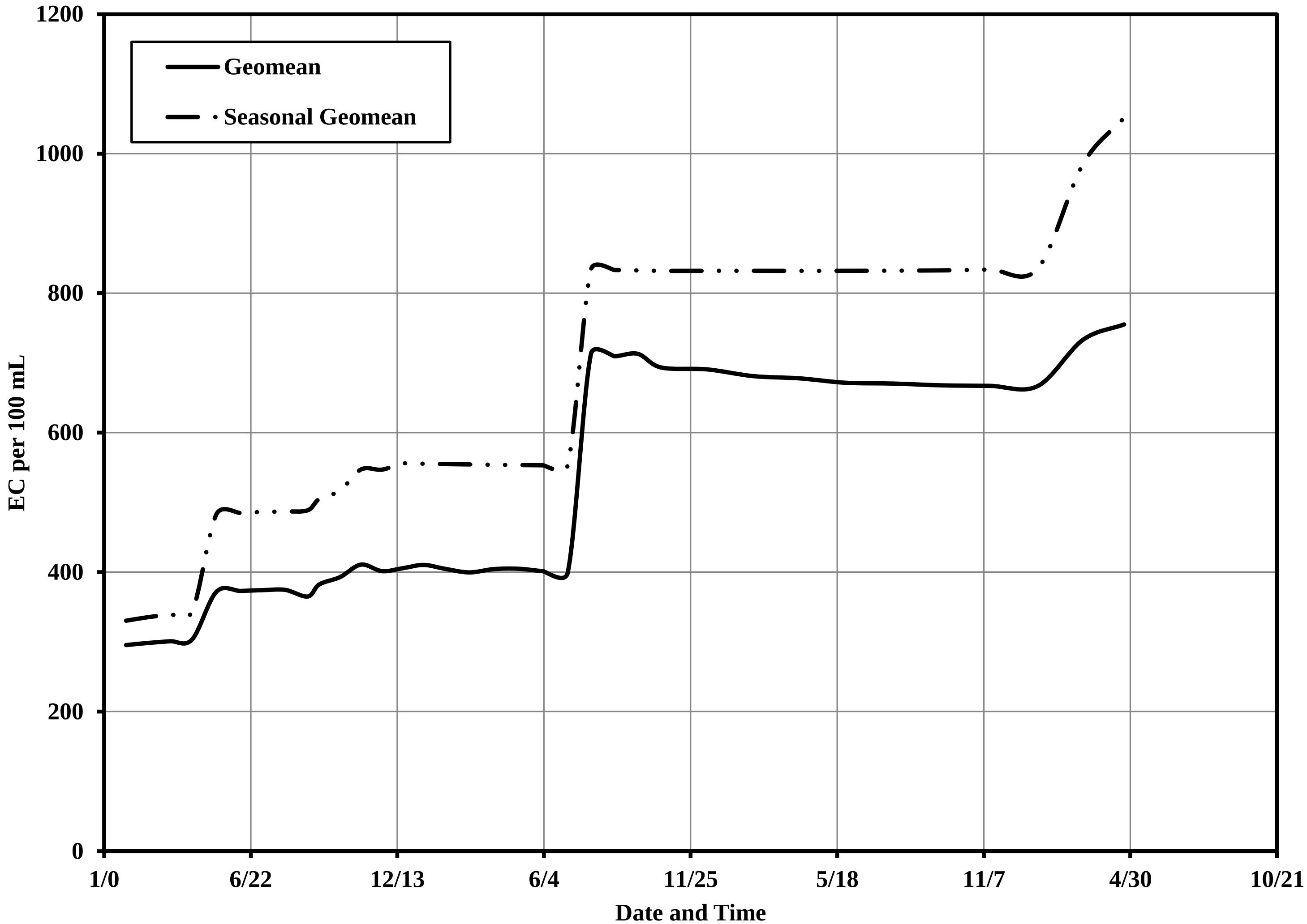

Figure IV-6 – Predicted Annual and Seasonal GM by Segments (2011)

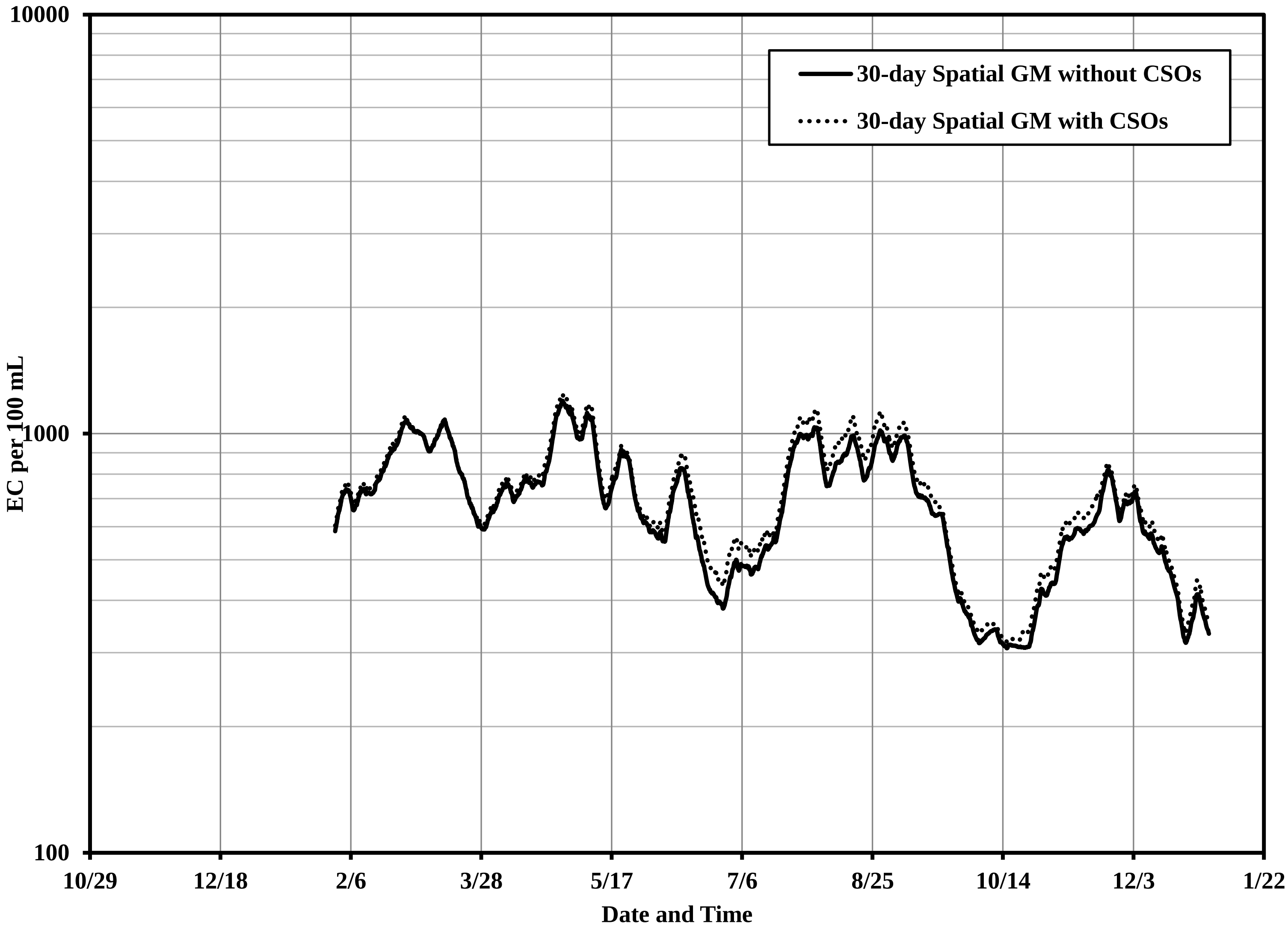

Figure IV-7 – Effect of CSOs on the Spatially Averaged Simulated GM - 2009

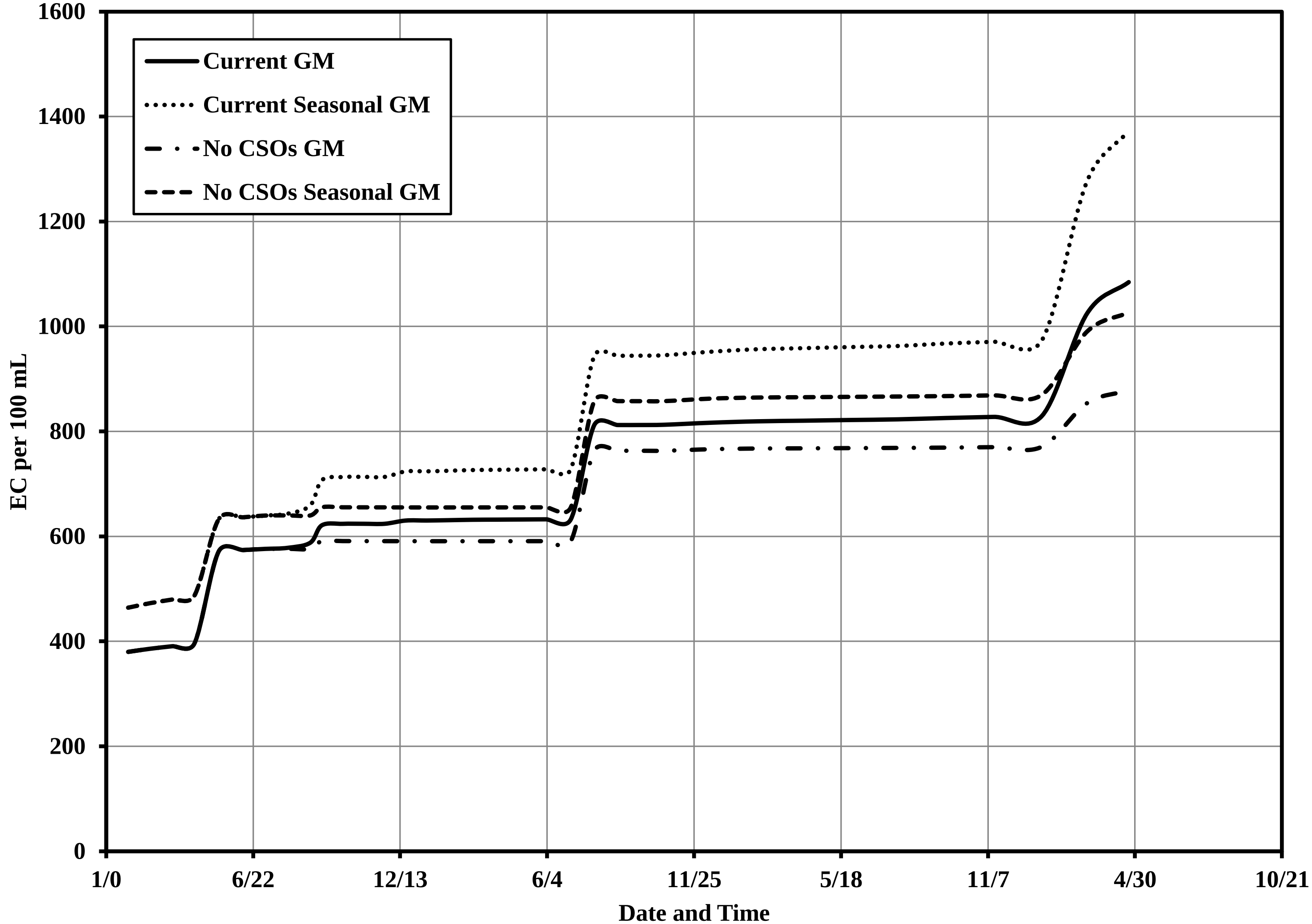

Figure IV-8 – Effect of CSOs on the Annual and Seasonal GM by Segment – 2009

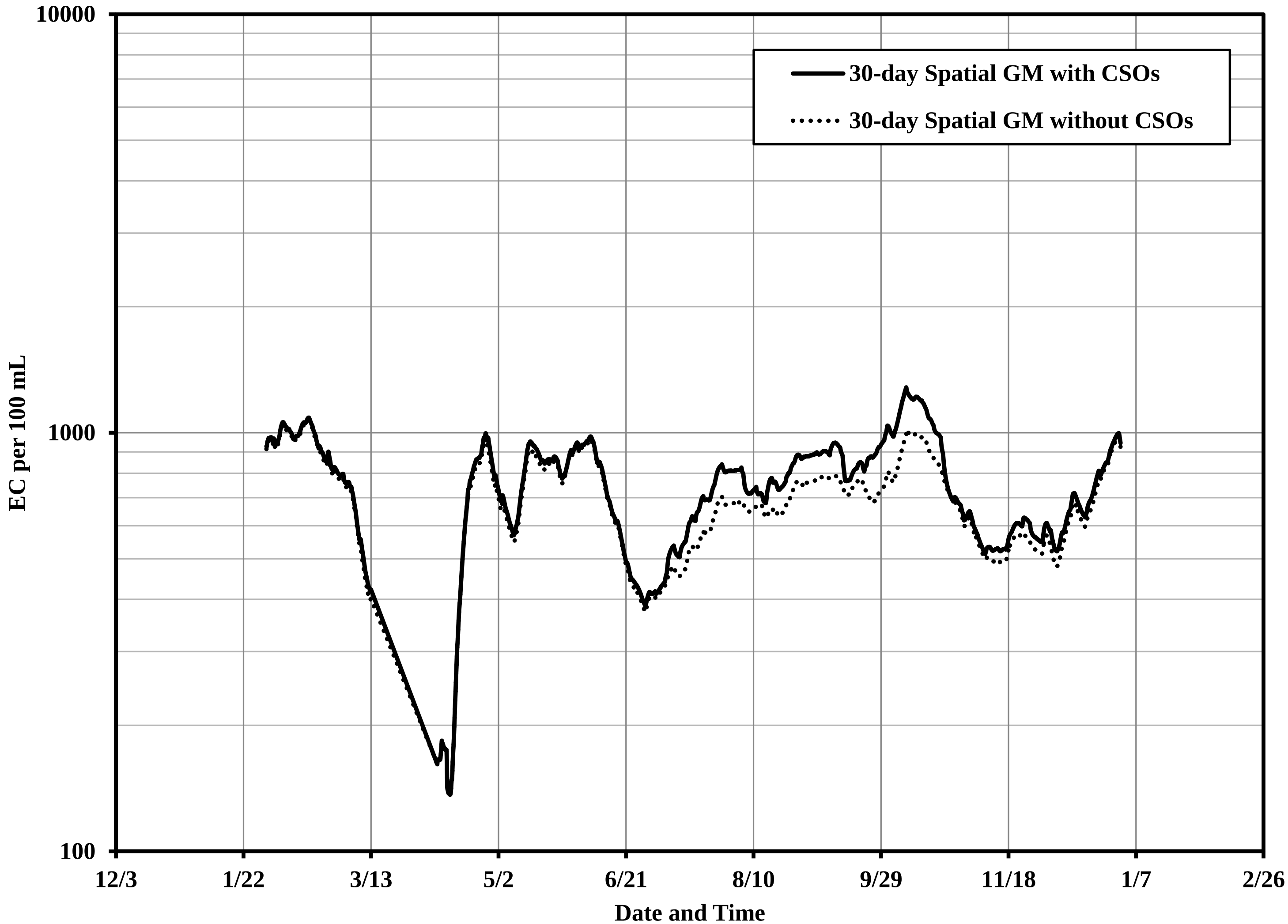

Figure IV-9 – Effect of CSOs on the Spatially Averaged Simulated GM – 2010

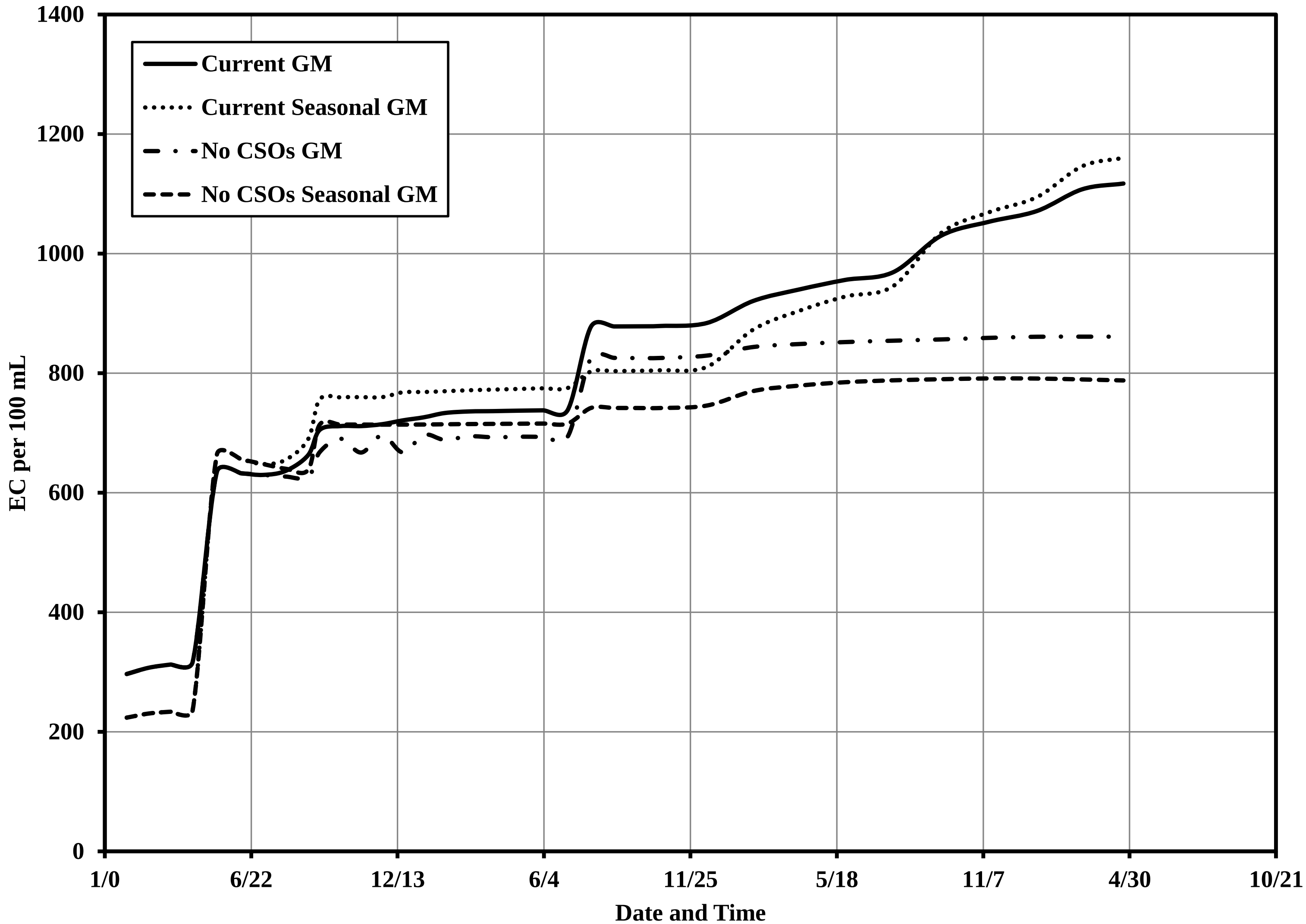

Figure IV-10 – Effect of CSOs on the Annual and Seasonal GM by Segments – 2010

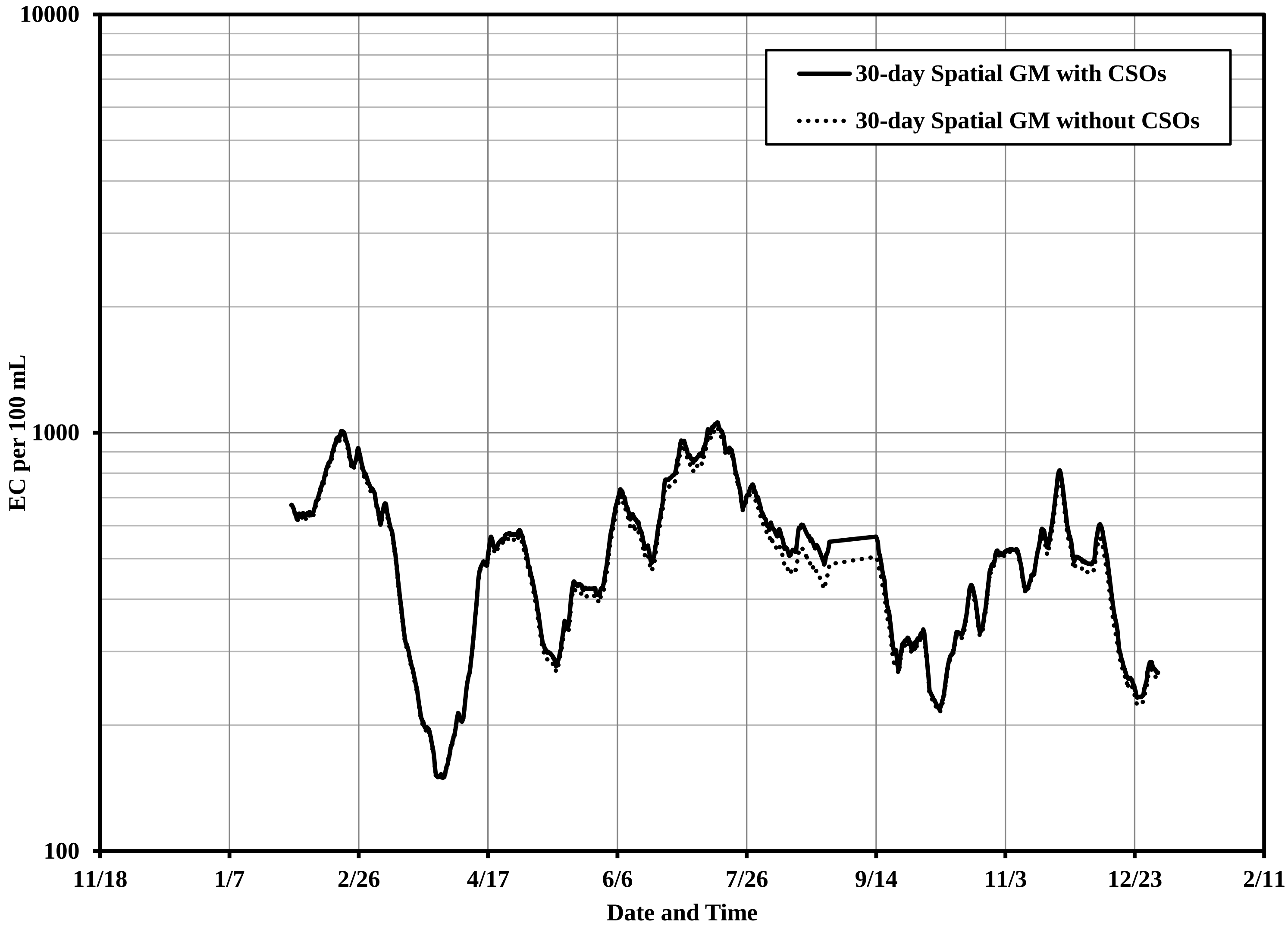

Figure IV-11 – Effect of CSOs on the Spatially Averaged Simulated GM – 2011

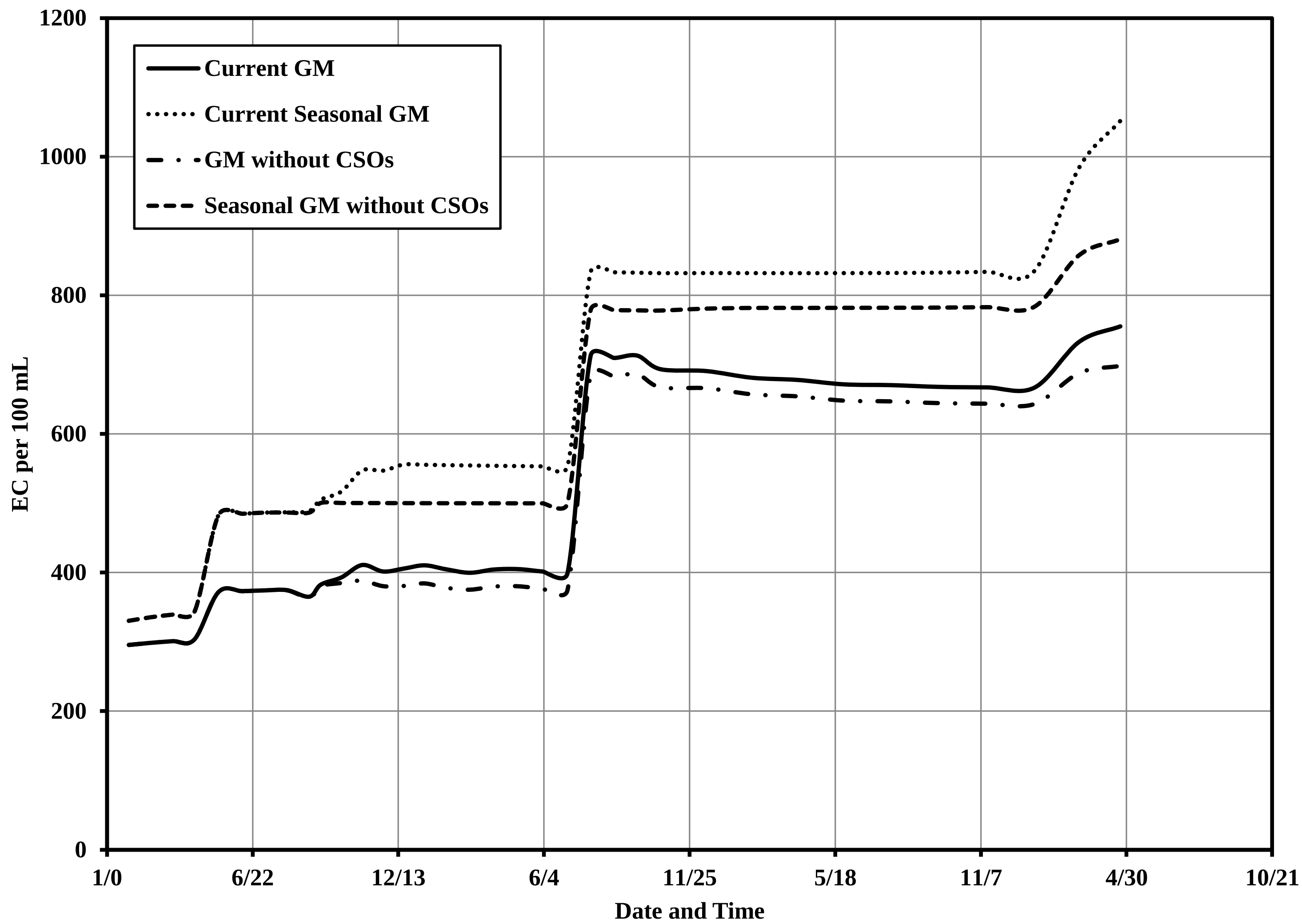

Figure IV-12 – Effect of CSOs on the Annual and Seasonal GM by Segment – 2011

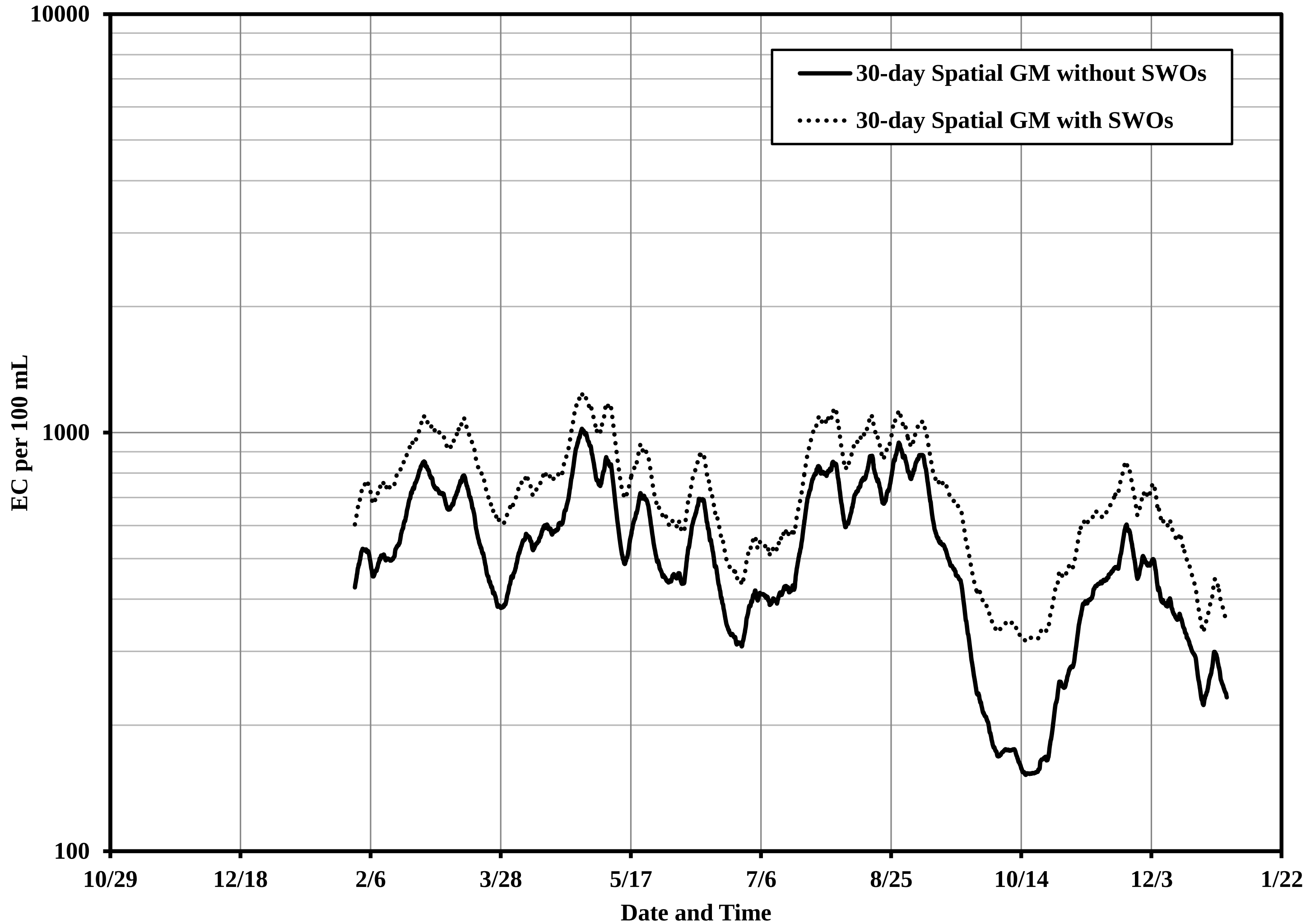

Figure IV-13 – Effect of SWOs on the Spatially Averaged Simulated GM – 2009

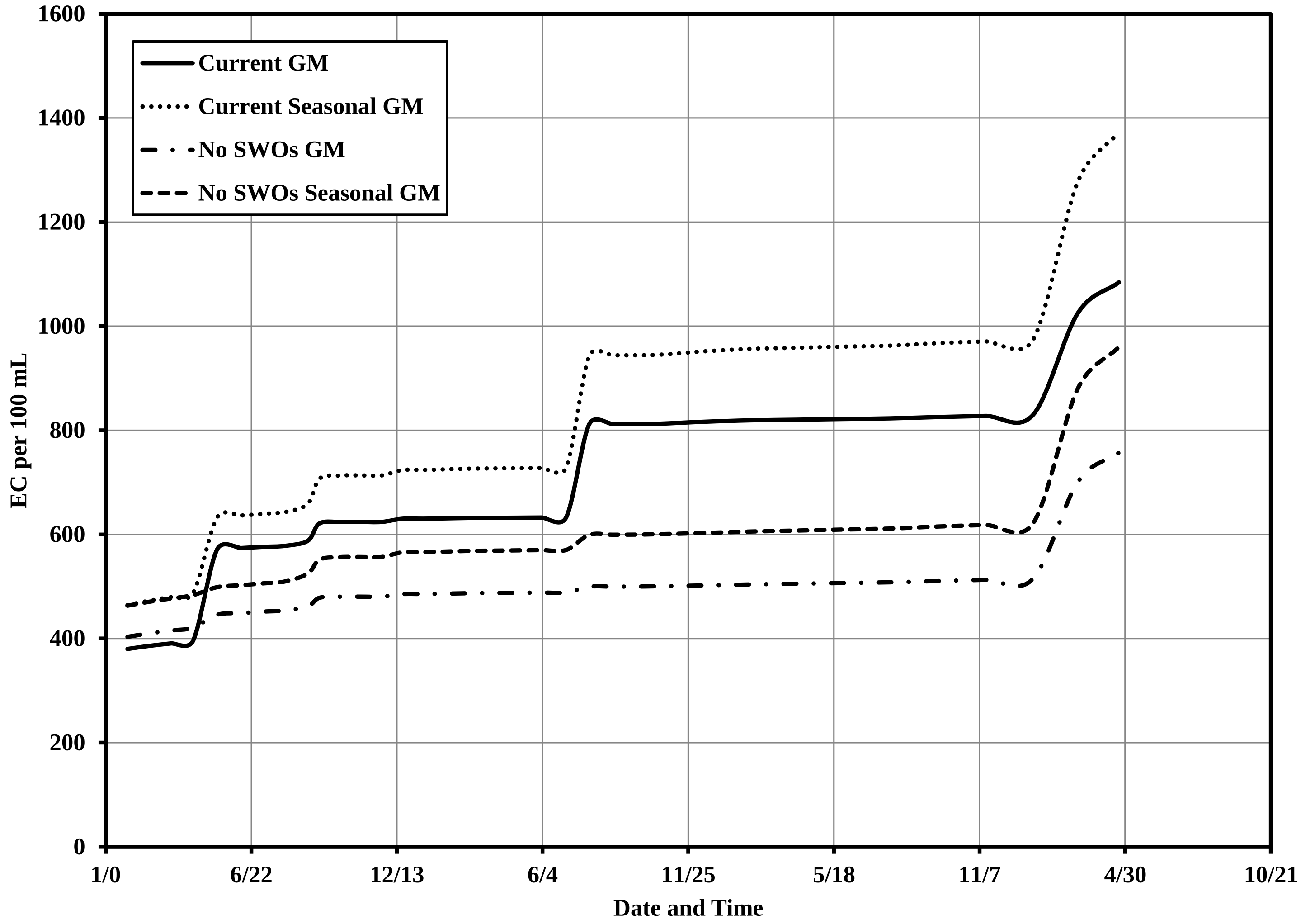

Figure IV-14 – Effect of SWOs on the Annual and Seasonal GM by Segment 2009

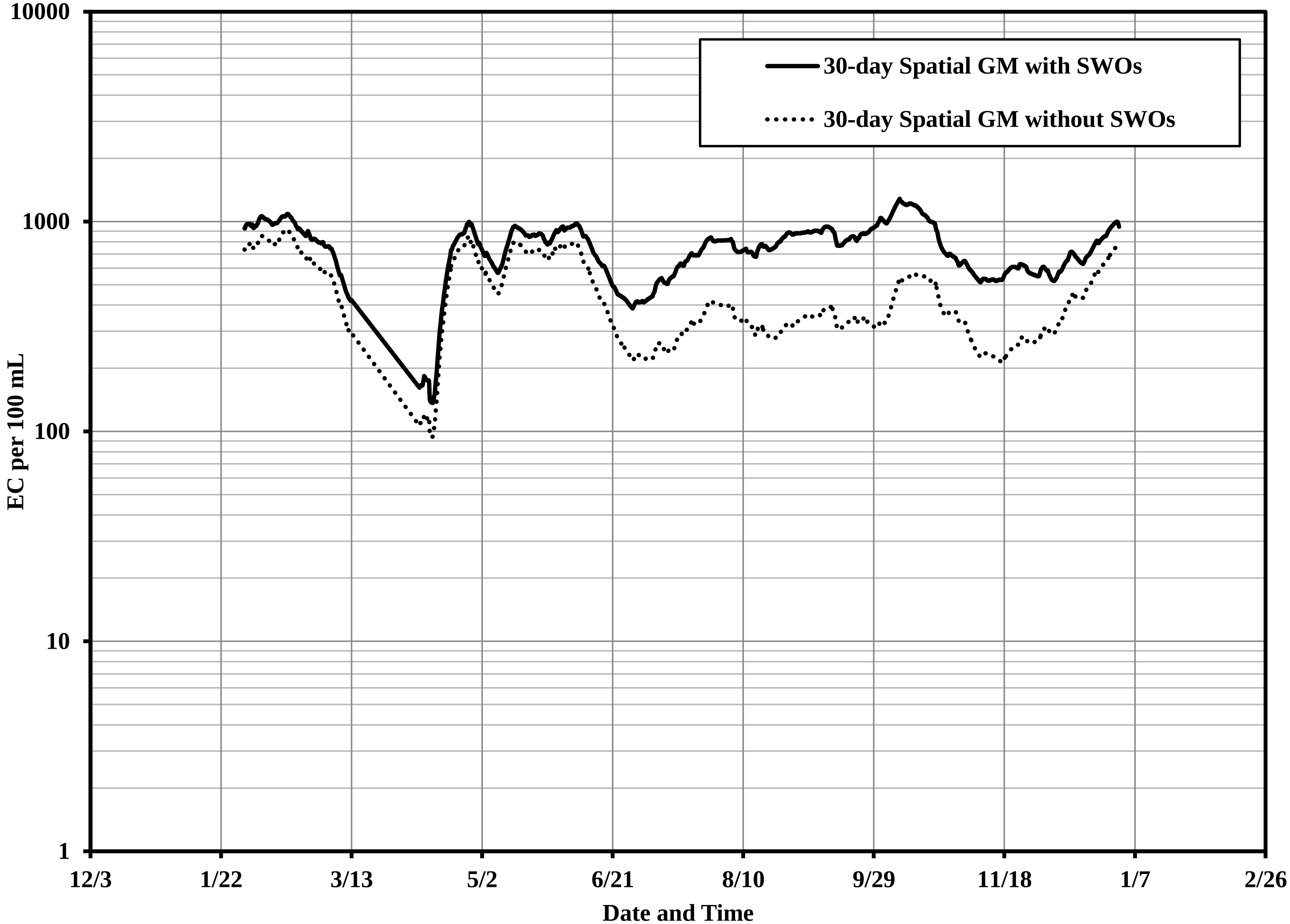

Figure IV-15 – Effect of SWOs on the Spatially Averaged Simulated GM – 2010

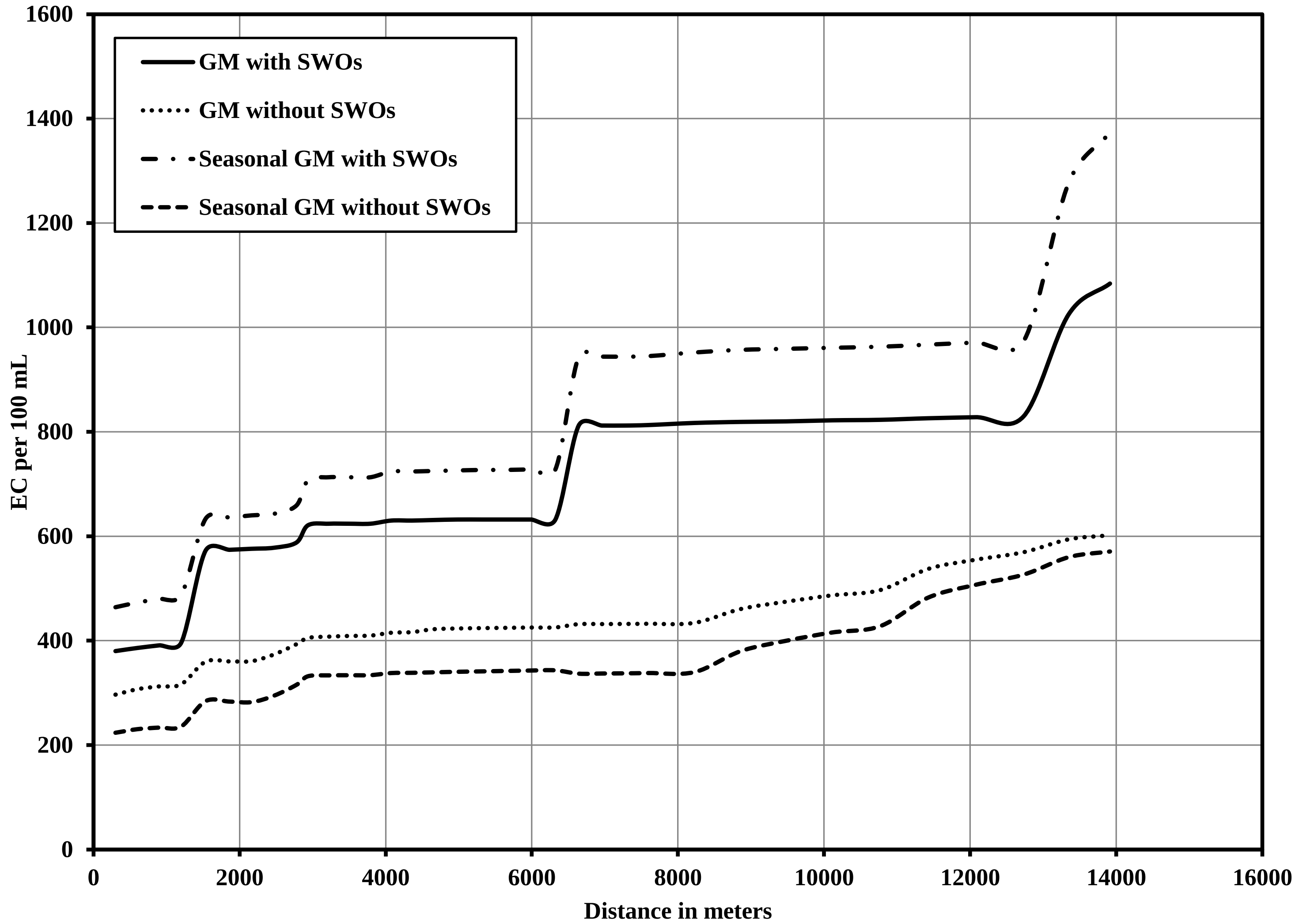

Figure IV-16 – Effect of SWOs on the Annual and Seasonal Simulated GM – 2010

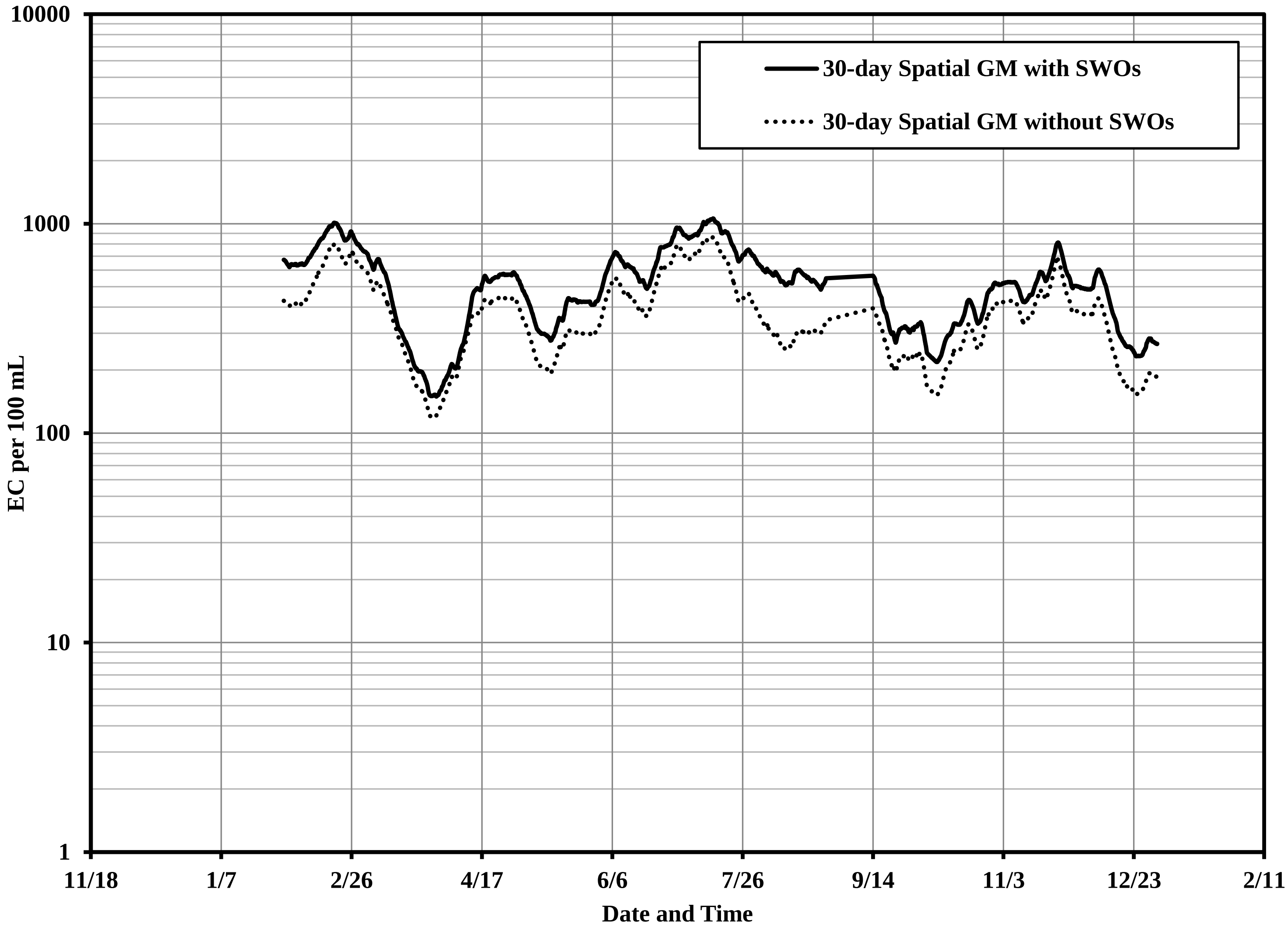

Figure IV-17 – Effect of SWOs on the Spatially Averaged Simulated GM – 2011

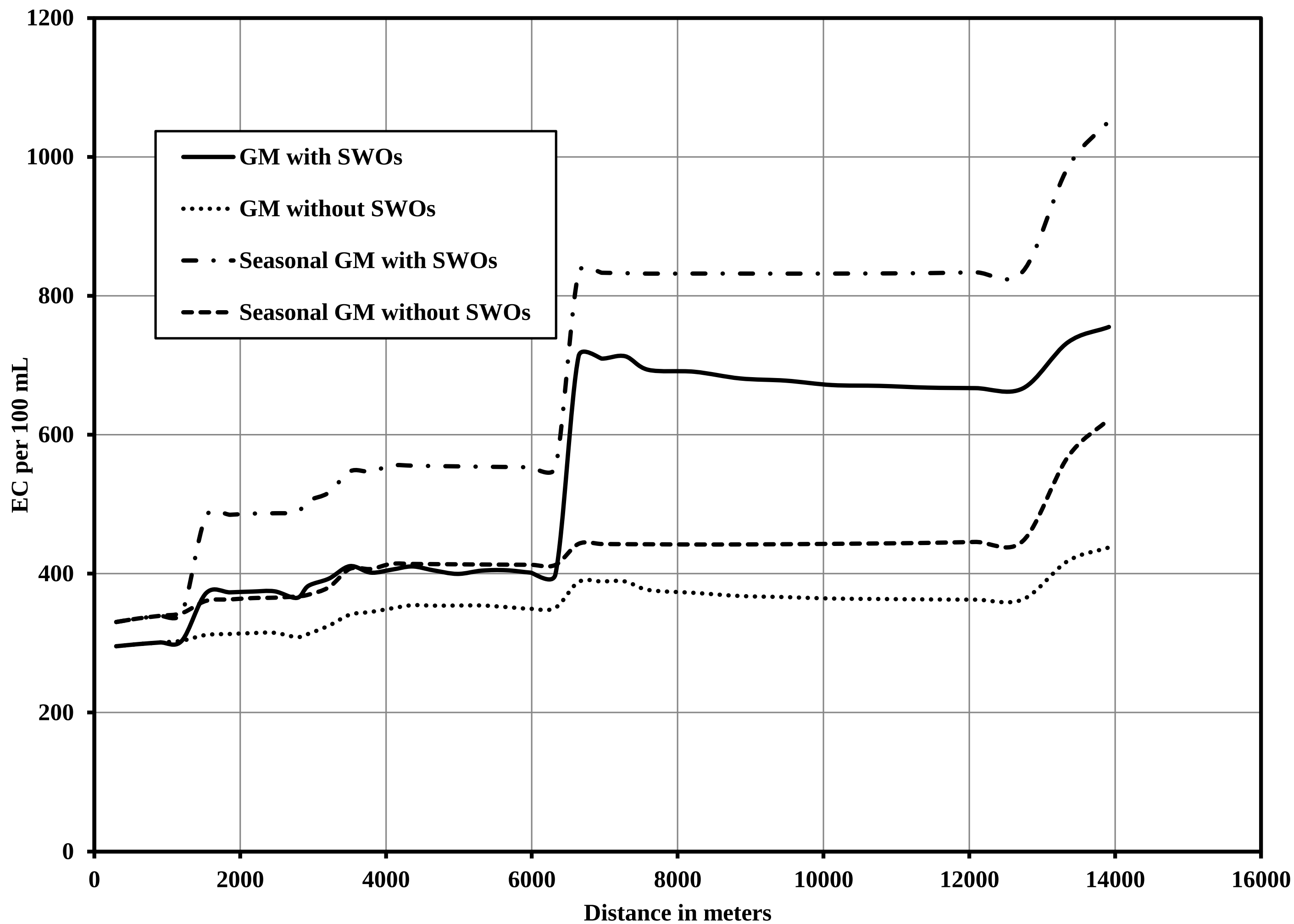

Figure IV-18 – Effect of SWOs on the Annual and Seasonal Simulated GM – 2011

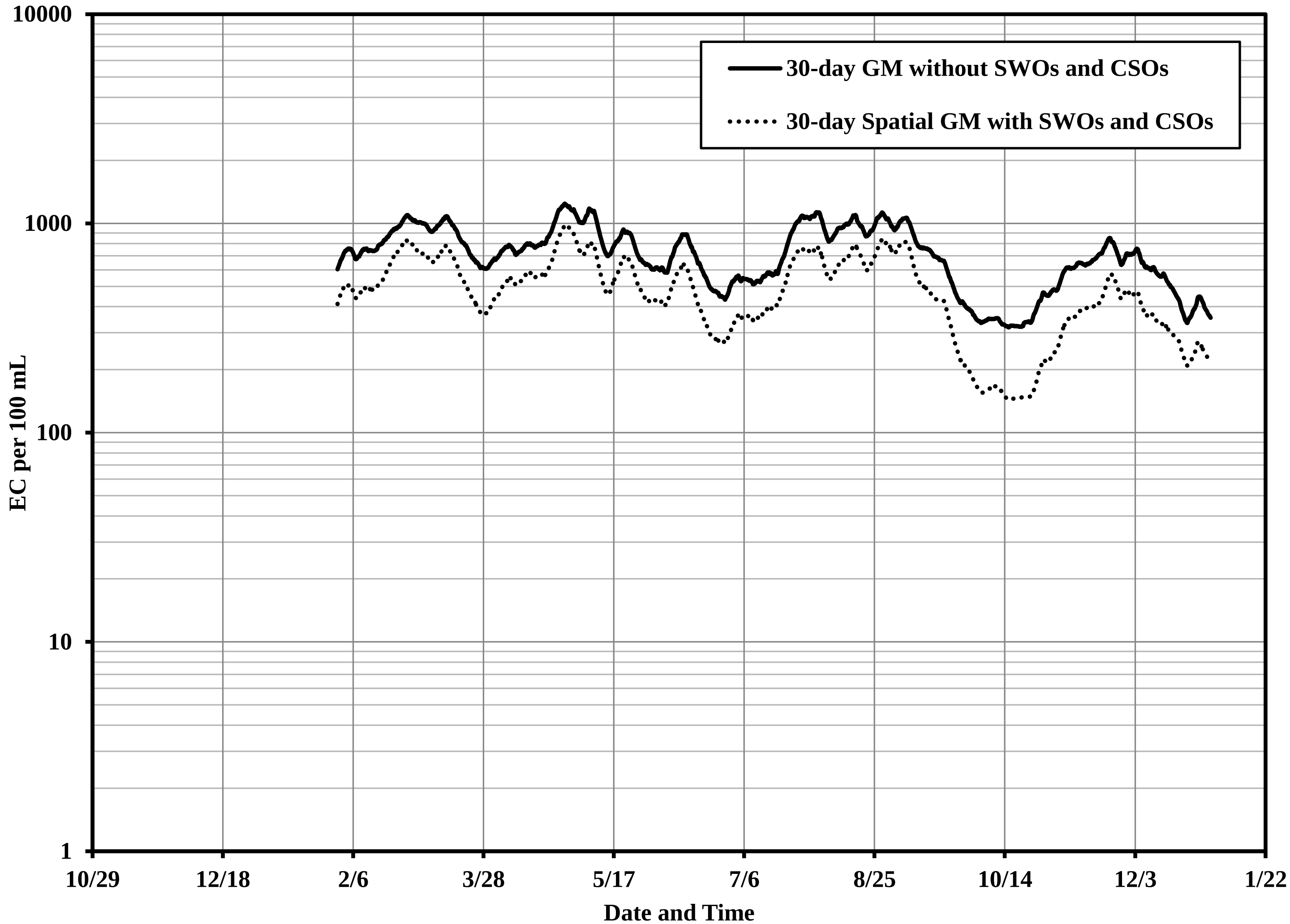

Figure IV-19 – Effect of CSOs and SWOs on the Spatially Averaged Simulated GM – 2009

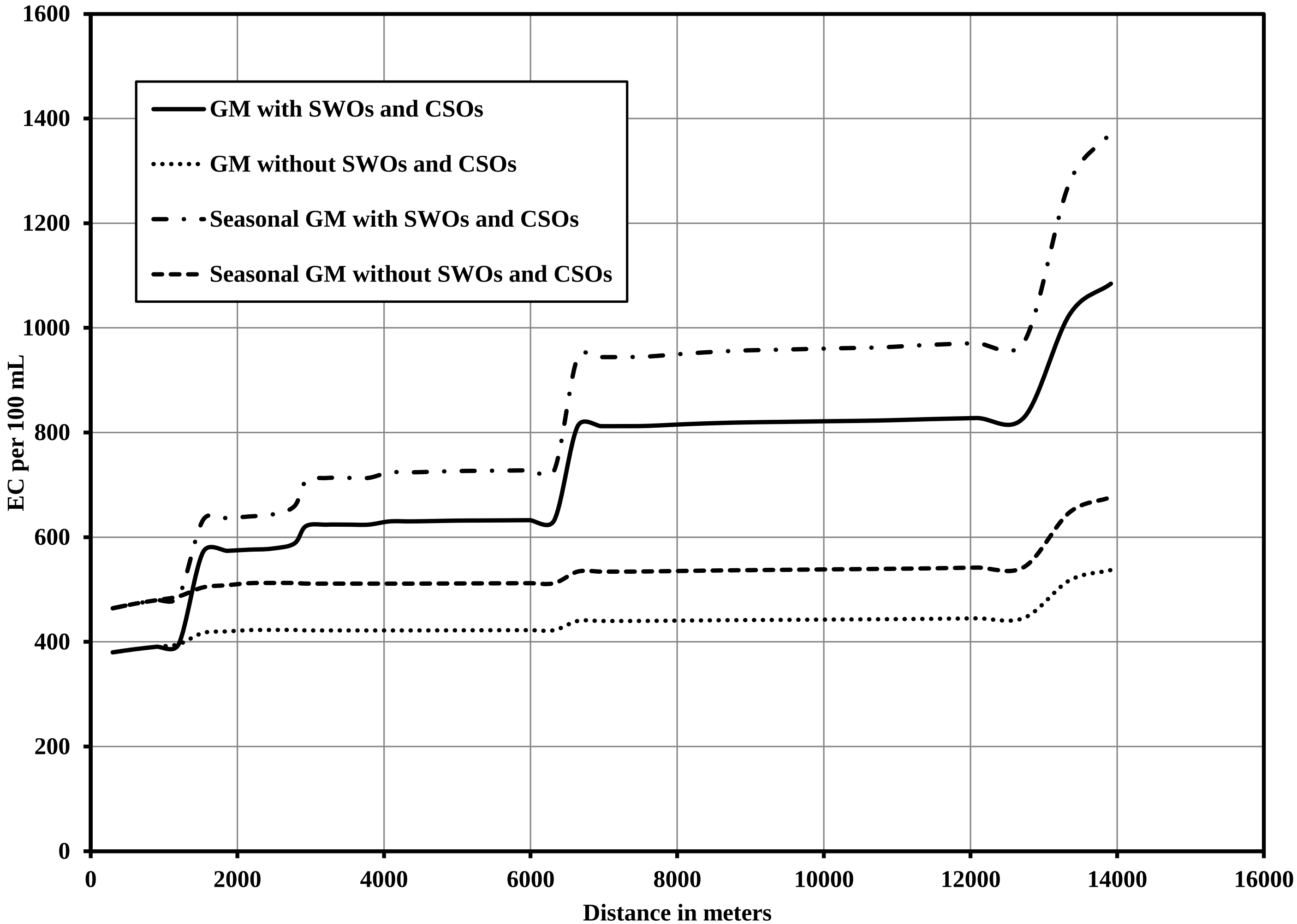

Figure IV-20 – Effect of CSOs and SWOs on the Annual and Seasonal Simulated GM – 2009

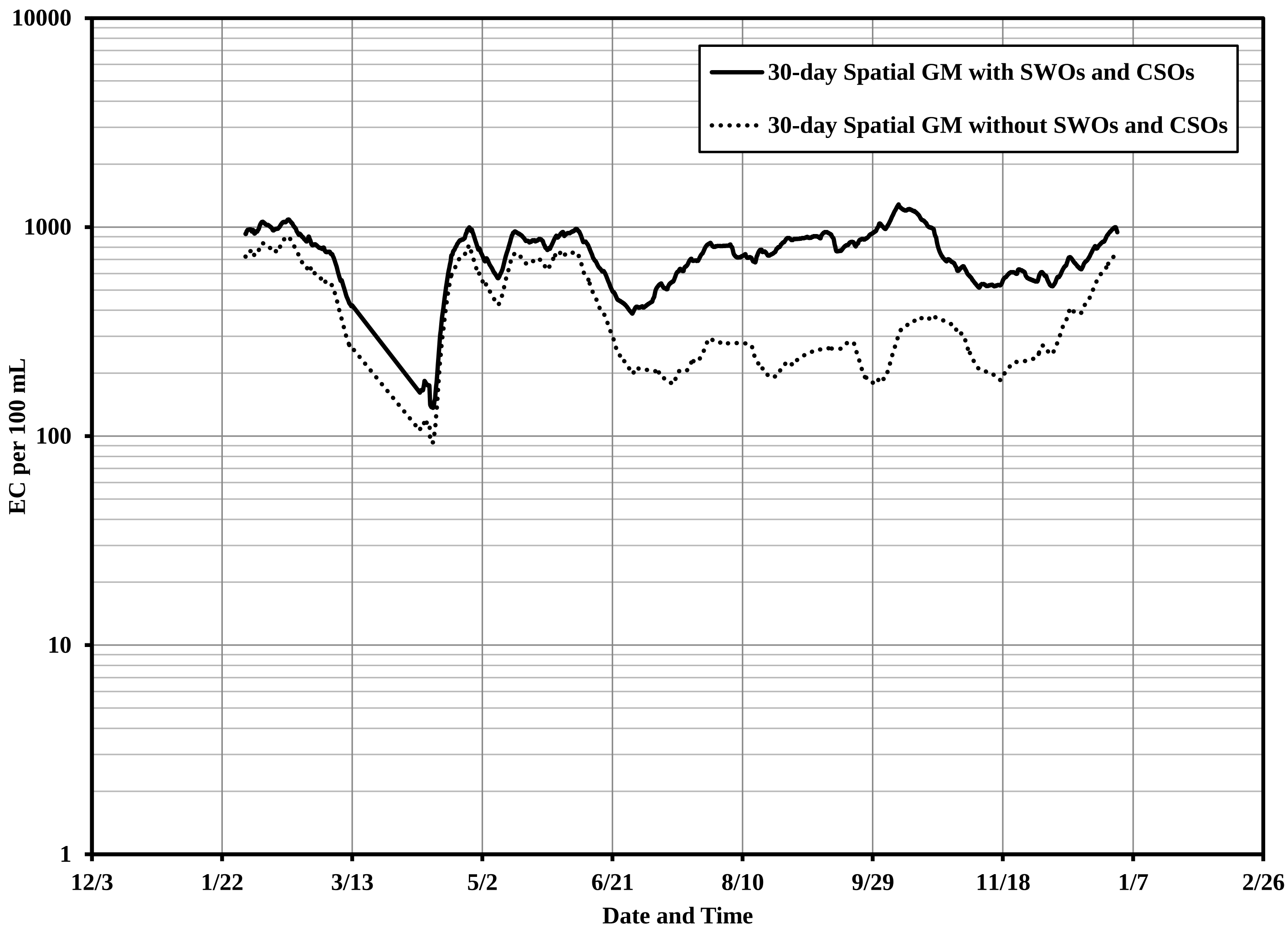

Figure IV-21 – Effect of CSOs and SWOs on the Spatially Averaged Simulated GM – 2010

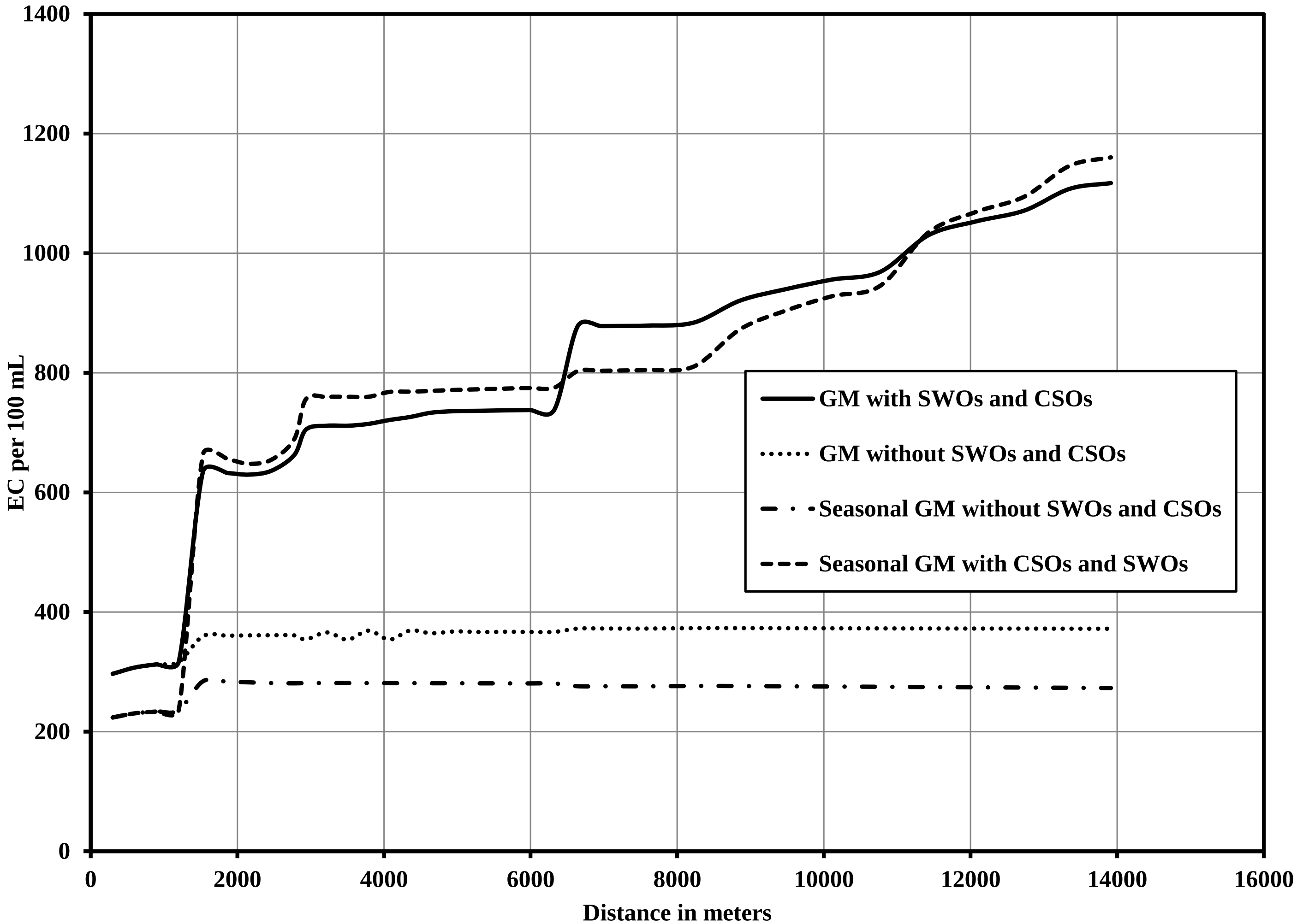

Figure IV-22 – Effect of CSOs and SWOs on the Annual and Seasonal Simulated GM – 2010

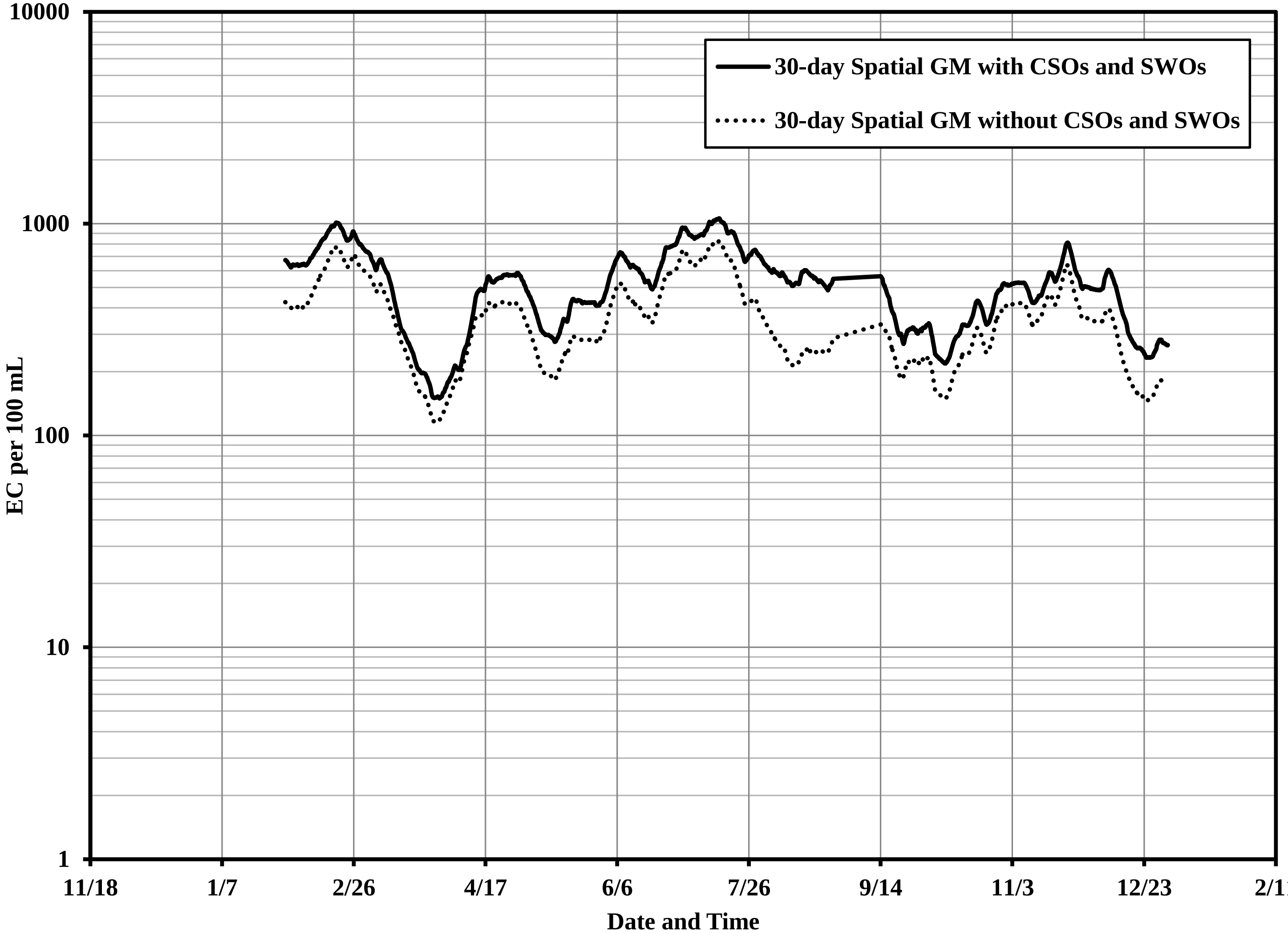

Figure IV-23 – Effect of CSOs and SWOs on the Spatially Averaged Simulated GM – 2011

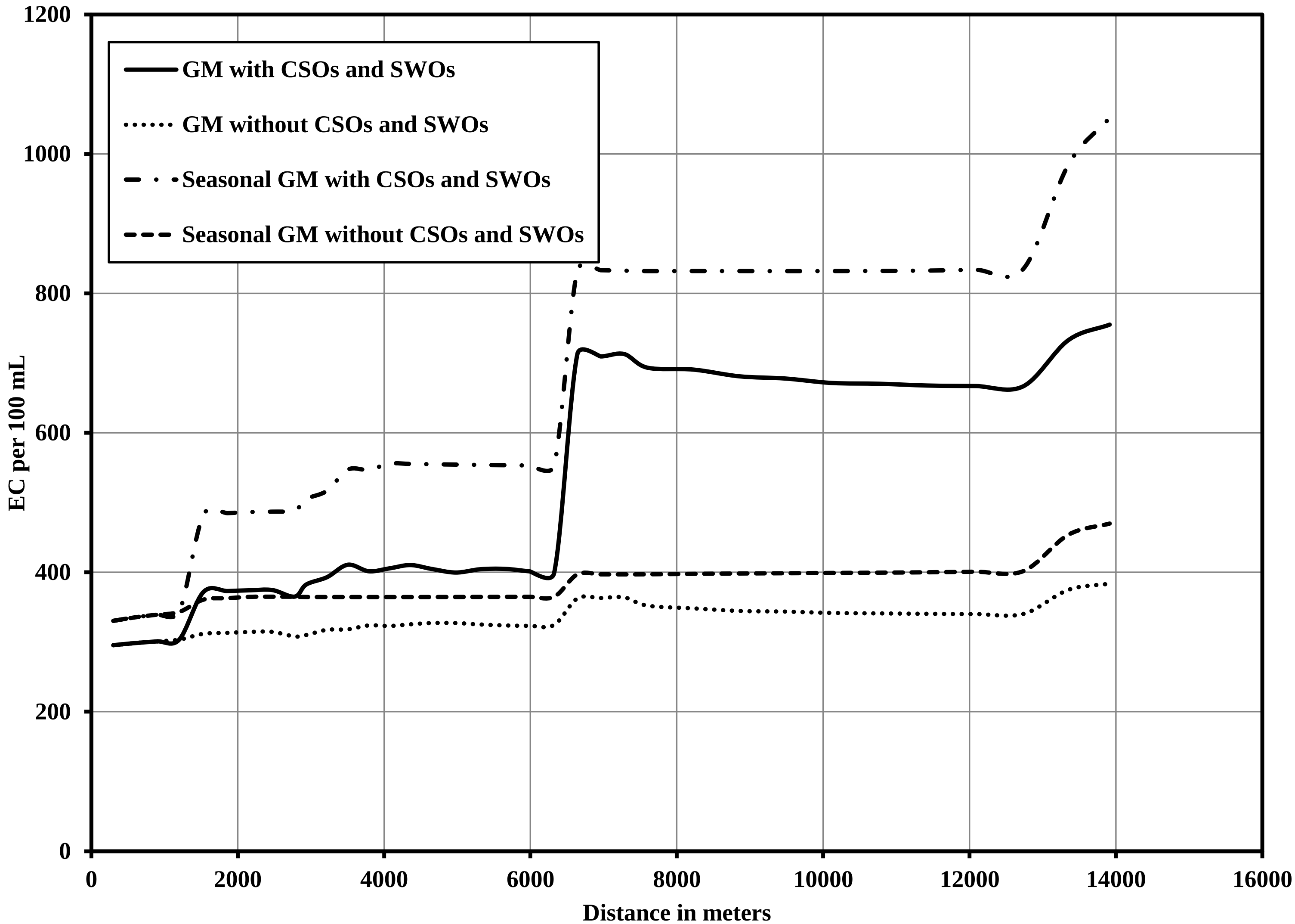

Figure IV-24 – Effect of CSOs and SWOs on the Annual and Seasonal Simulated GM – 2011

Table IV-1 – Effect of Sources

			Eliminate CSOs		90% reduction in SWO Concentration		Both forcings	
			GM	**% red**	**GM**	**% red**	**GM**	**% red**
2009	Spatially averaged	Annual	627	5	489	26	430	35
		Seasonal	709	8	574	26	523	32
	Down-stream	Annual	878	19	757	30	537	50
		Seasonal	1027	25	960	30	676	51
2010	Spatially averaged	Annual	672	7	423	36	360	50
		Seasonal	638	10	345	55	272	62
	Down-stream	Annual	861	23	602	44	372	67
		Seasonal	788	32	571	58	273	76
2011	Spatially averaged	Annual	461	3	348	27	330	31
		Seasonal	572	6	412	32	377	38
	Down-stream	Annual	698	8	438	42	384	49
		Seasonal	881	16	622	41	470	55

V. TMDL Calculations

A. TMDL Objectives

To compute the total maximum daily load, the requirement was set such that the seasonal (May 15 to September 15) geometric mean is less than the geometric mean target of 126 EC per 100 mL based on New Jersey State Water Quality Standards (NJDEP, 2009) of the year 2008 as recommended by the NJDEP.

B. Summary of TMDL Condition

For the water quality standards to be met the following conditions are to be applied simultaneously,

1) The CSOs must be eliminated
2) Storm water and tributary concentrations must be rolled back by 90%
3) The upstream boundary must be rolled back by 75%

C. Margin of Safety

The TMDL condition summarized above has a 50% probability of attaining the water quality standards, if implemented. We propose to rollback the upstream boundary more to obtain a 95% probability of attaining the standards.

We observe that the error is normally distributed with a standard deviation 0.64 and the number of samples is 197. Therefore, the log average of the concentrations has a standard error given by

$$Std. Error = \frac{0.64}{\sqrt{196}} = 0.046$$

Therefore, to be 95% confident that the standard is met, the rollback target must be,

$$Rollback\ Target = 10^{\log 126 - Z_{0.95} 0.046} = 106$$

This results in an additional rollback on the upstream boundary by 5%. Therefore, with MOS, the upstream must be rolled back by 80%.

D. TMDL Allocations

For the above TMDL condition, The TMDL allocation must be

1) WLA - 0
2) LA – 90% below current levels
3) Upstream boundary must be rolled back by 75% without MOS and 80% with MOS.

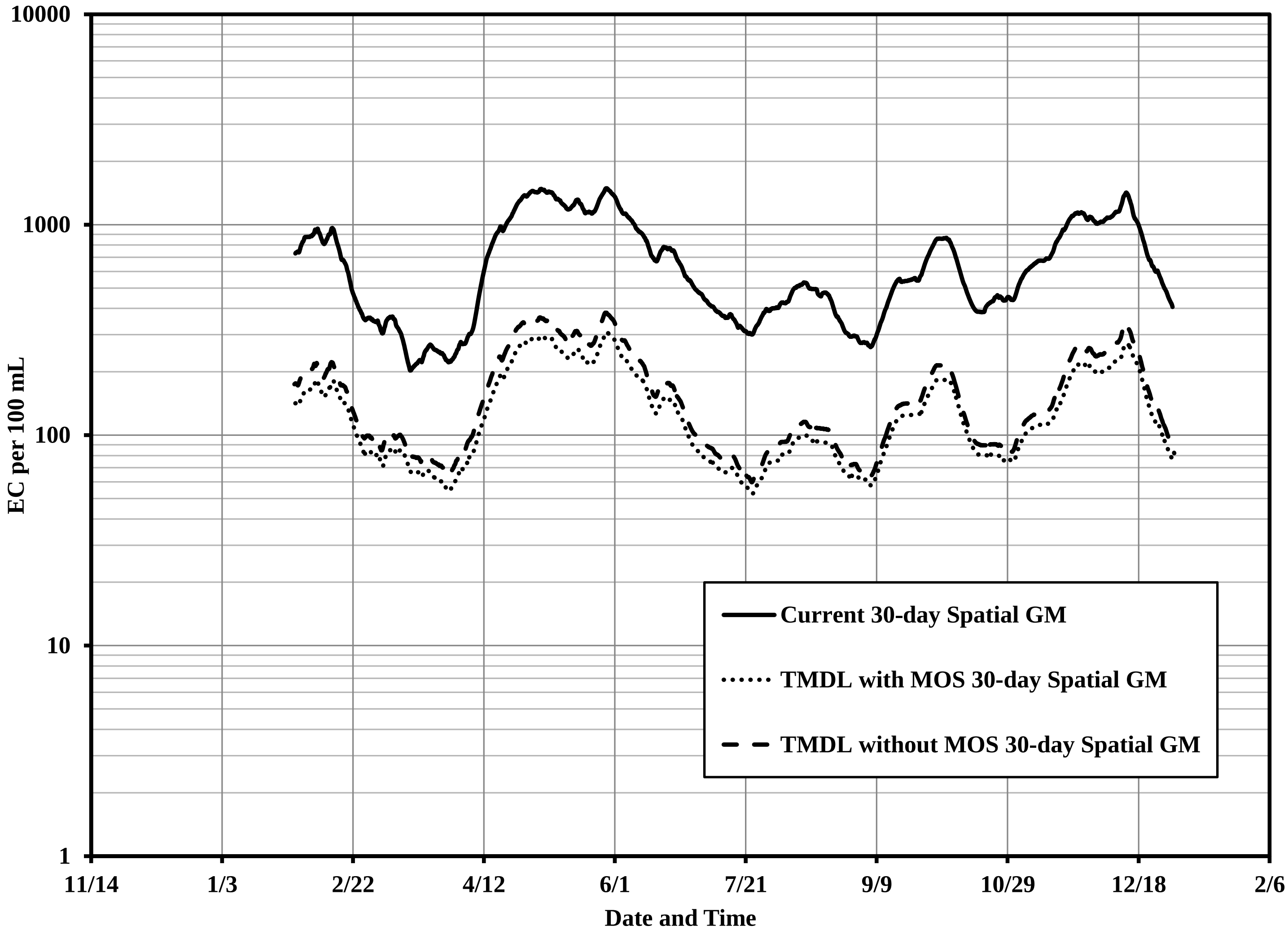

Figure V-1 – Spatially Averaged GM for Current and TMDL Conditions (2008)

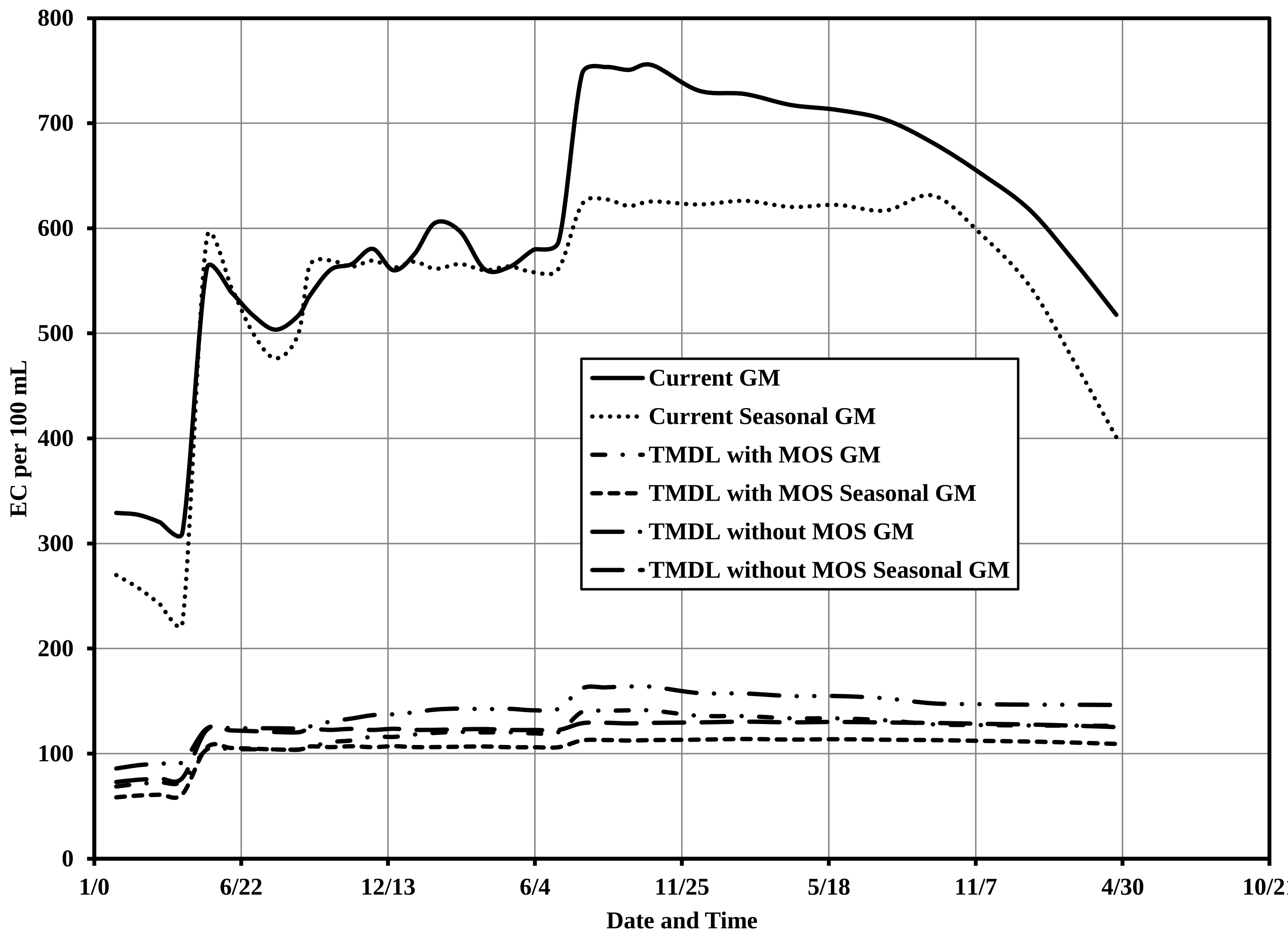

Figure V-2 – Annual and Seasonal Spatial GM for Current and TMDL Conditions (2008)

VI. Conclusions

The following major conclusions can be drawn from the work,

1) Stormwater concentration reduction to 90% has much higher effect on decreasing the geometric mean compared to eliminating the CSOs

2) There is a high degree of variability in water quality among various years due to varying flow and meteorological forcings

3) Completely eliminating the CSOs and reducing the stormwater concentrations by 90% will improve water quality significantly. However, even these actions will not ensure water quality standards are met in this reach.

4) The only way to meet water quality standards is to improve upstream boundary water quality in addition to eliminating the CSOs and reducing storm water concentrations by 90%.

5) The required upstream boundary rollback is 40% without a margin of safety and 50% with a margin of safety (MOS). The MOS ensures the confidence of water compliance is 95%. If the MOS is not applied, the confidence of compliance is 50%.

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VIII. Time Series Plots

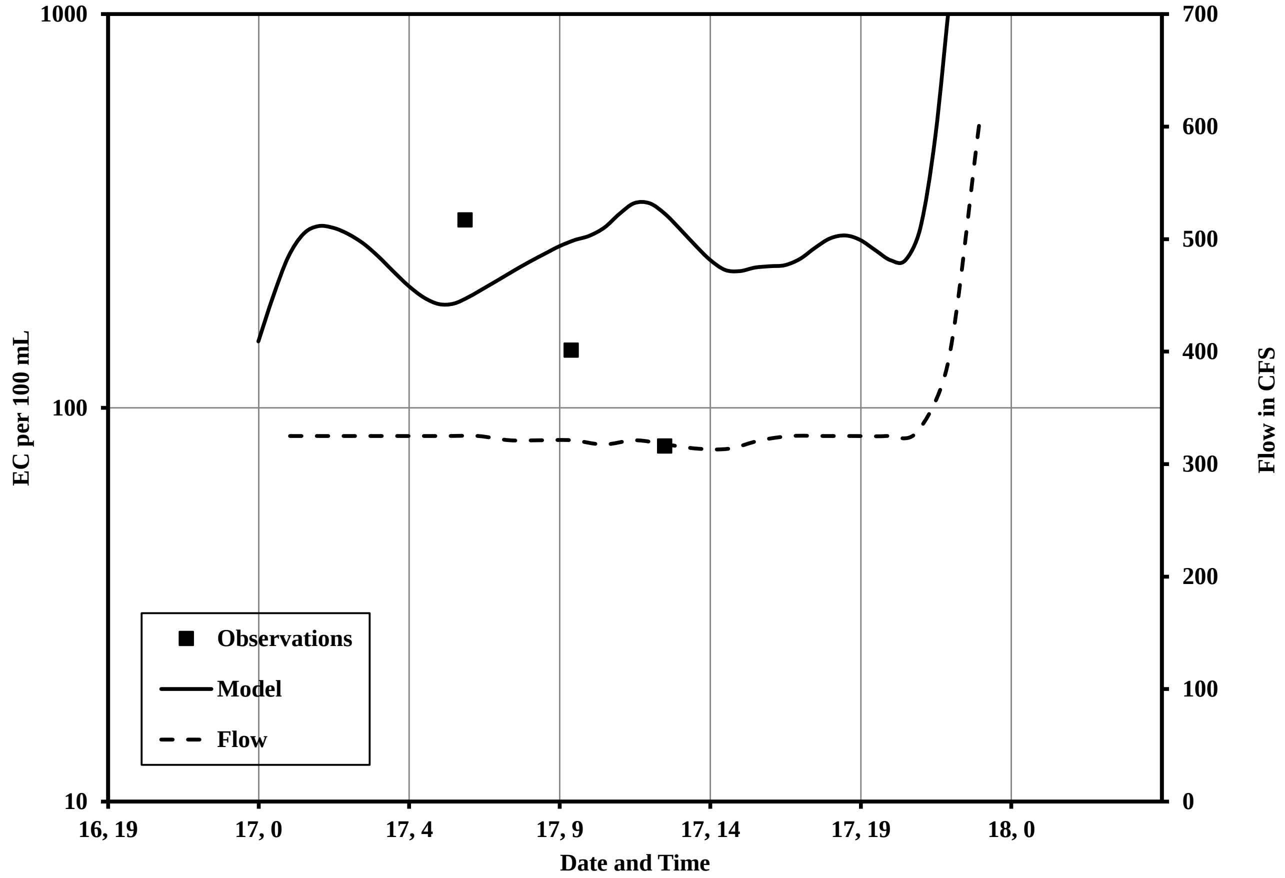

Figure VIII-1 - WAYNE DRY EVENT 1 JULY 17, 2009

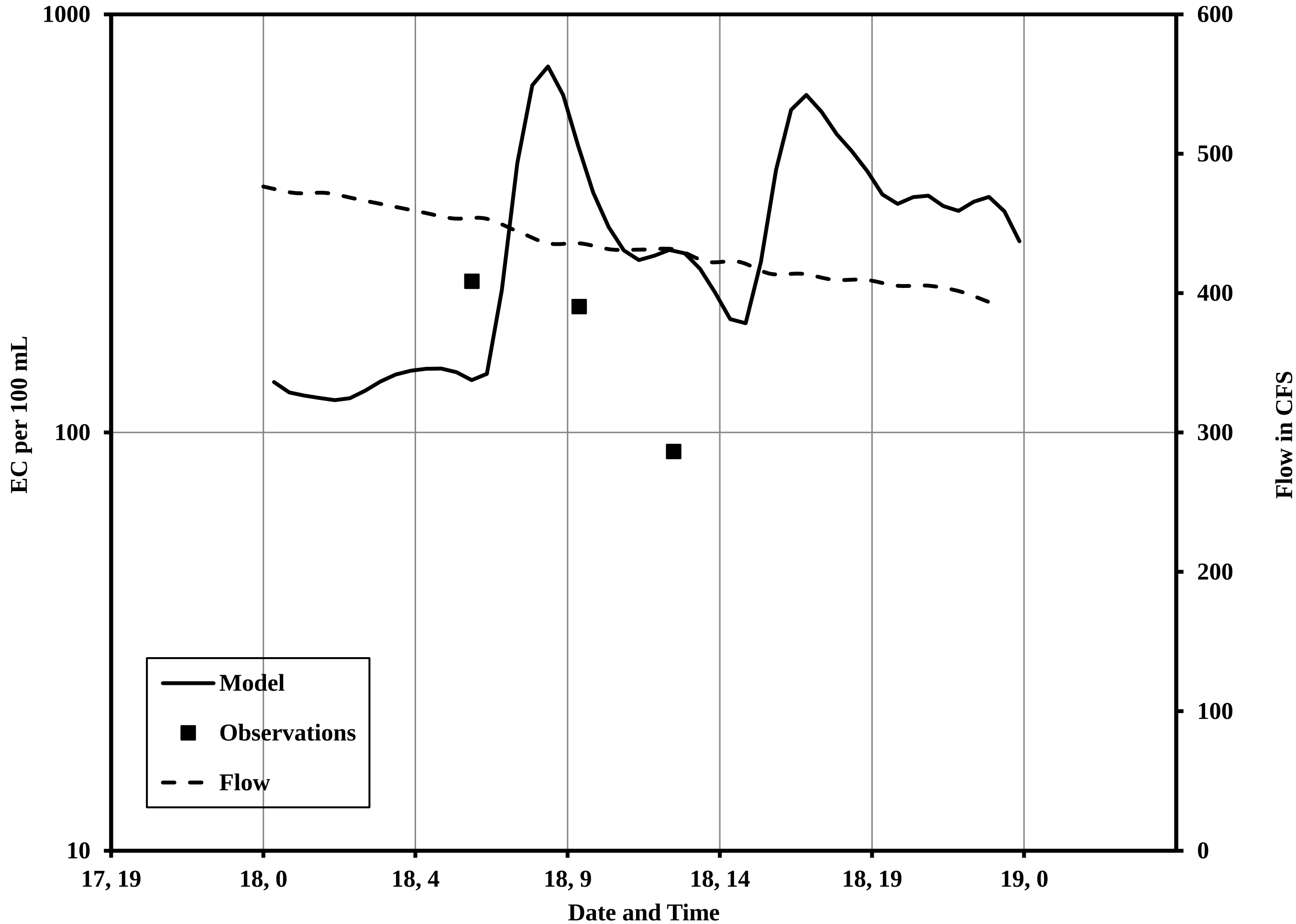

Figure VIII-2 - WAYNE DRY EVENT 2 AUGUST 18, 2009

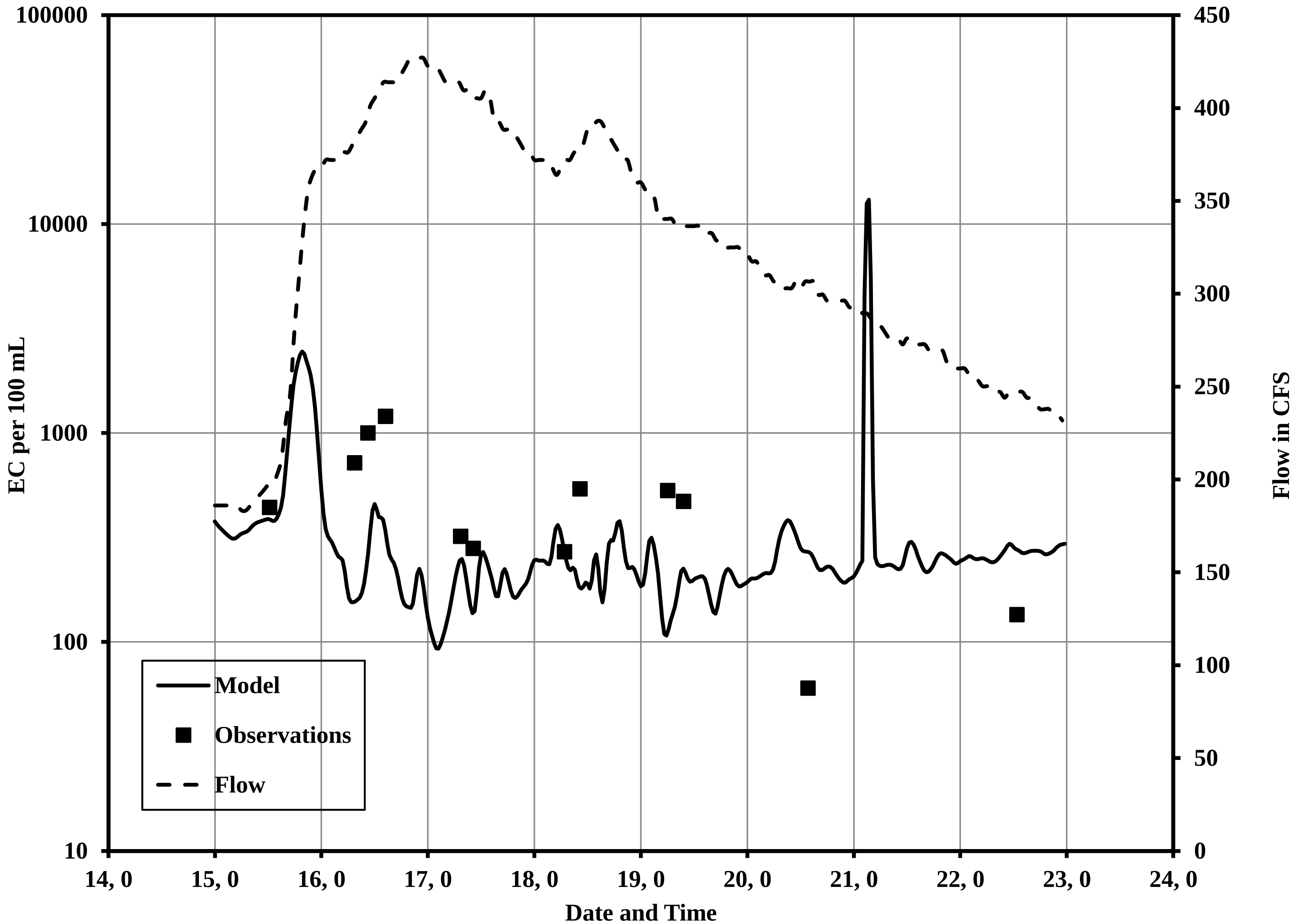

Figure VIII-3 - WAYNE WET EVENT 1 WITH FLOW OCTOBER 15-22, 2009

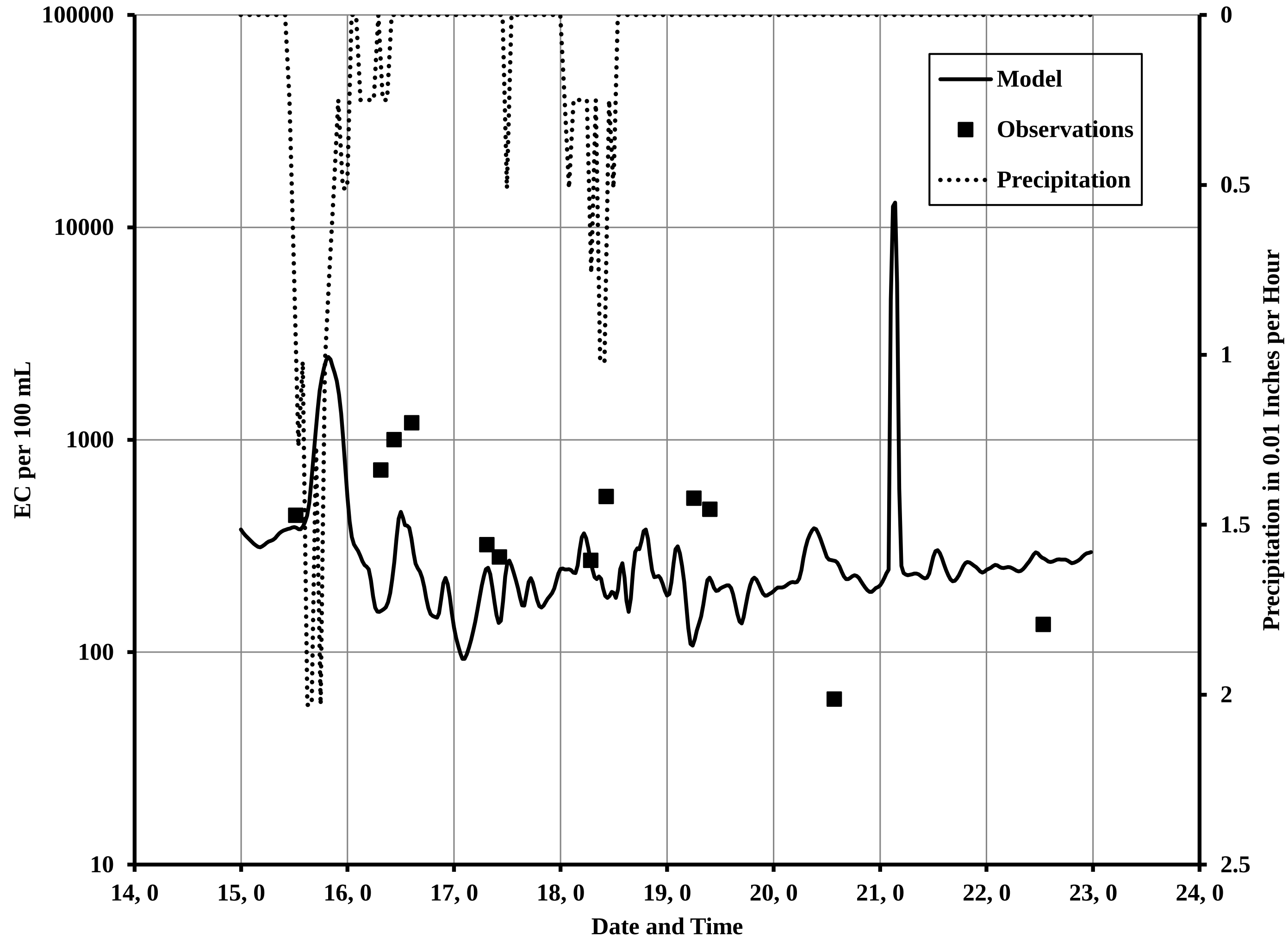

Figure VIII-4 - WAYNE WET EVENT 1 WITH PRECIPITATION OCTOBER 15-22, 2009

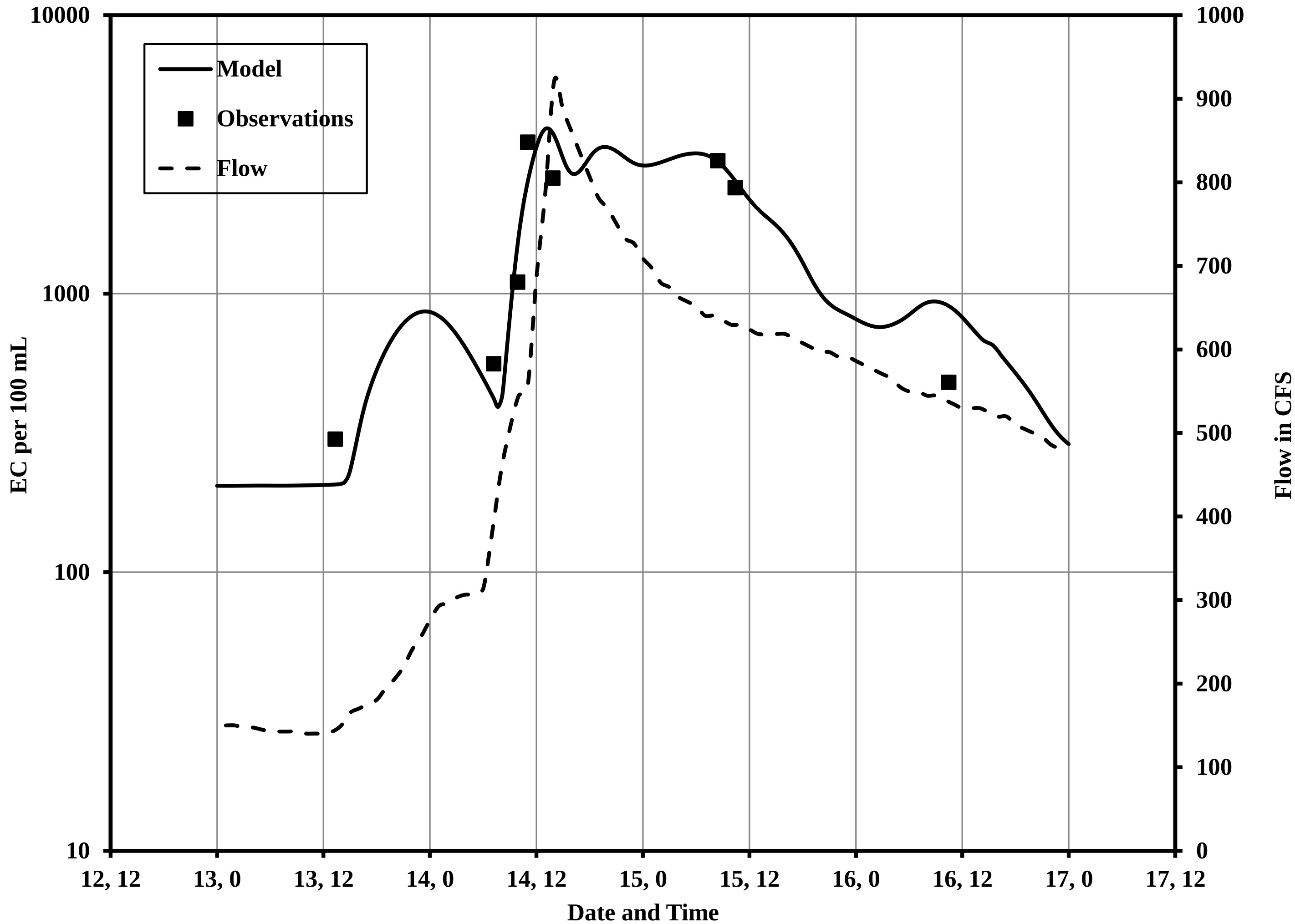

Figure VIII-5 - WAYNE WET EVENT 2 WITH FLOW JULY 13-16, 2010

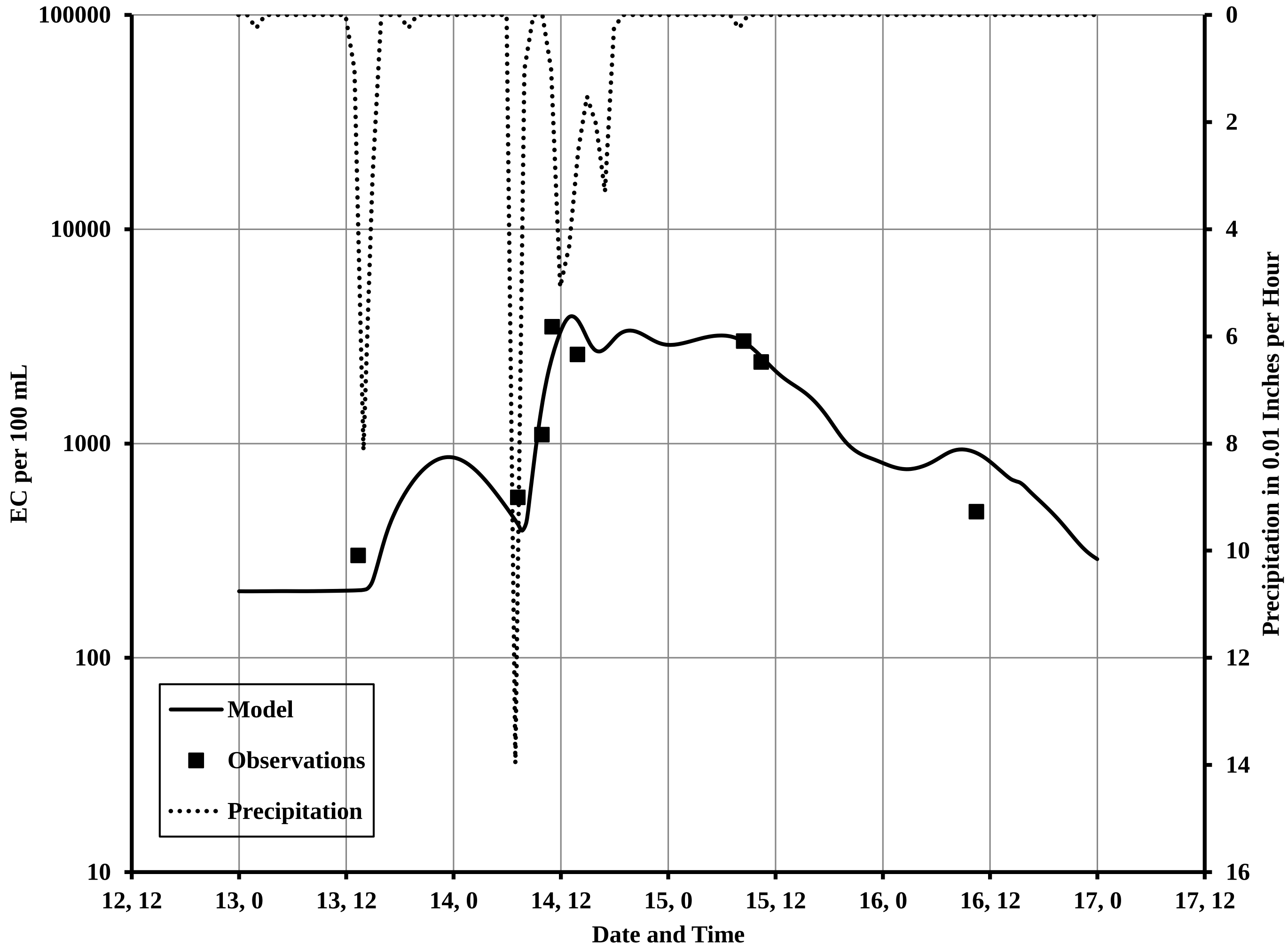

Figure VIII-6 - WAYNE WET EVENT 2 WITH PRECIPITATION JULY 13-16, 2010

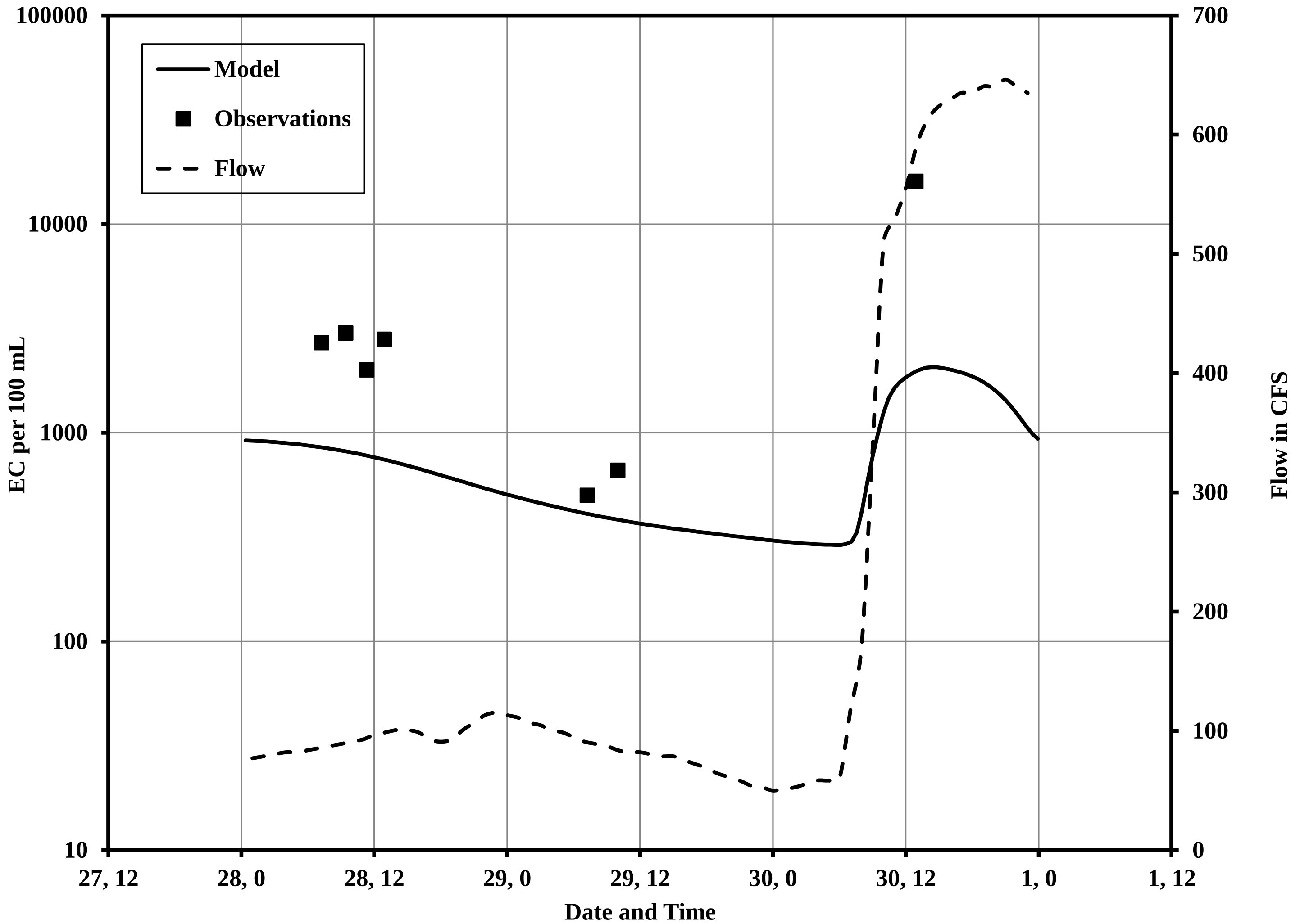

Figure VIII-7 – WAYNE WET EVENT 3 WITH FLOW OCTOBER 15, 28-30, 2010

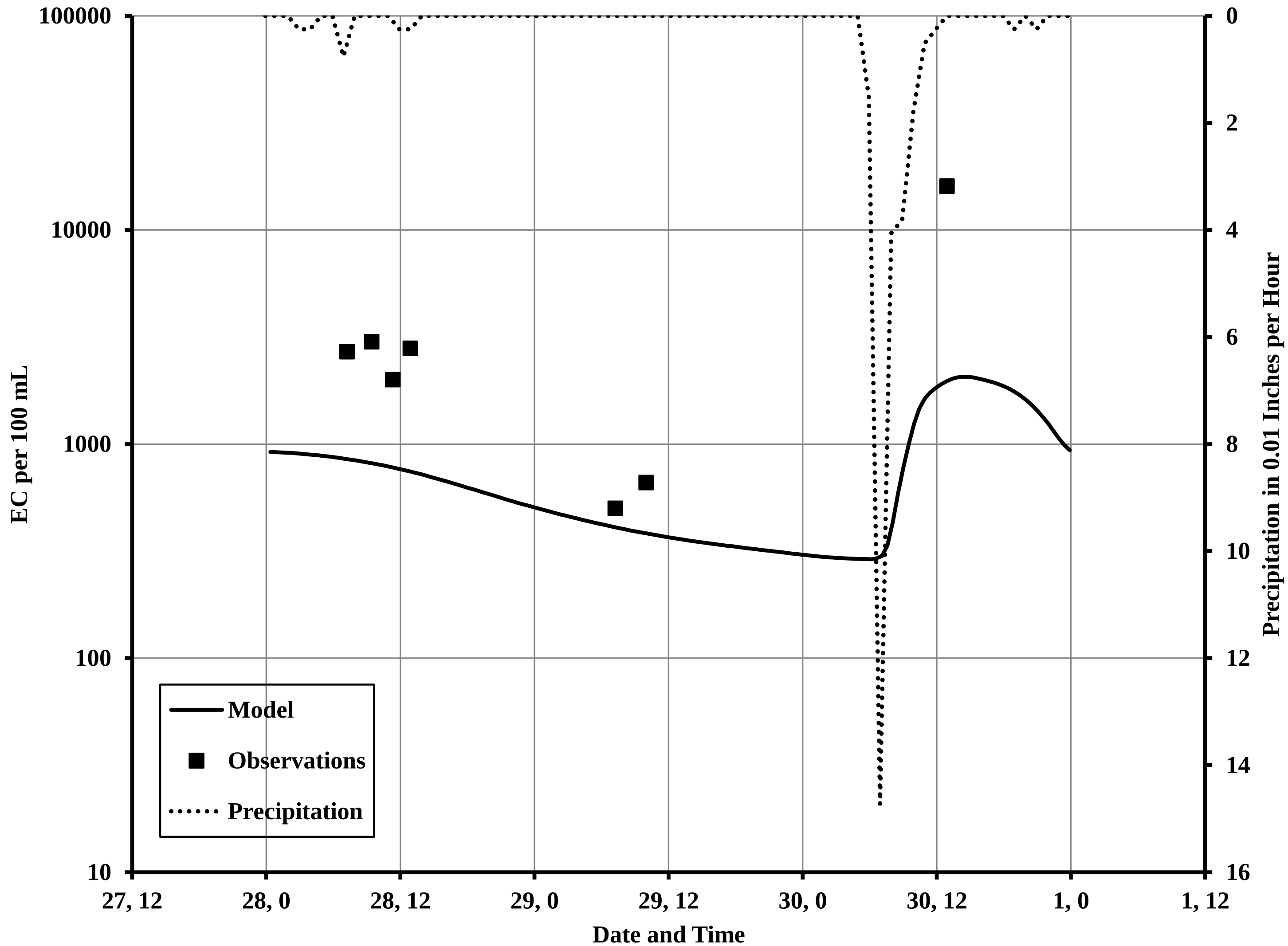

Figure VIII-8 – WAYNE WET EVENT 3 WITH PRECIPITATION OCTOBER 15, 28-30, 2010

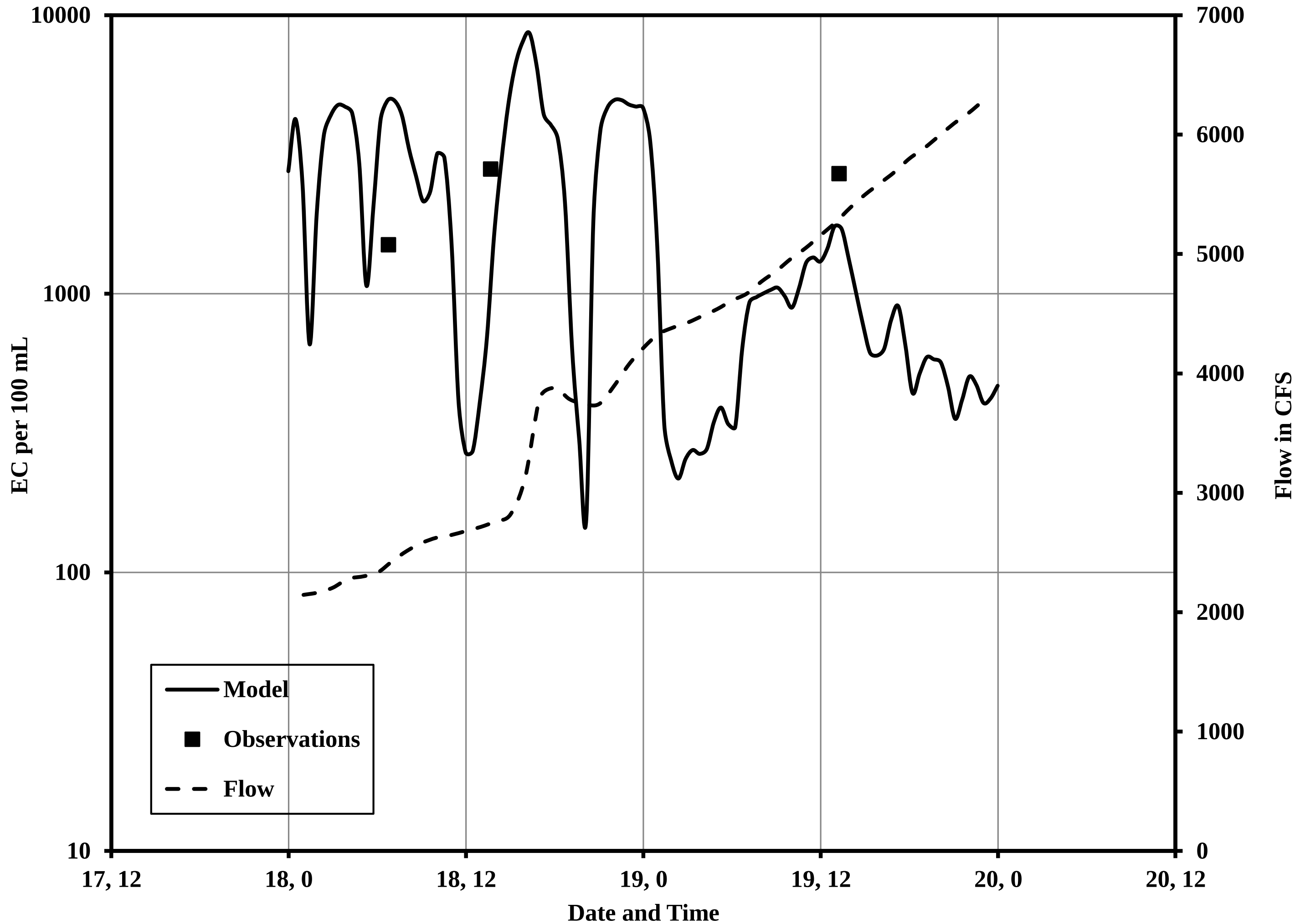

Figure VIII-9 - WAYNE WET EVENT 4&5 WITH FLOW MAY 18-19, 2011

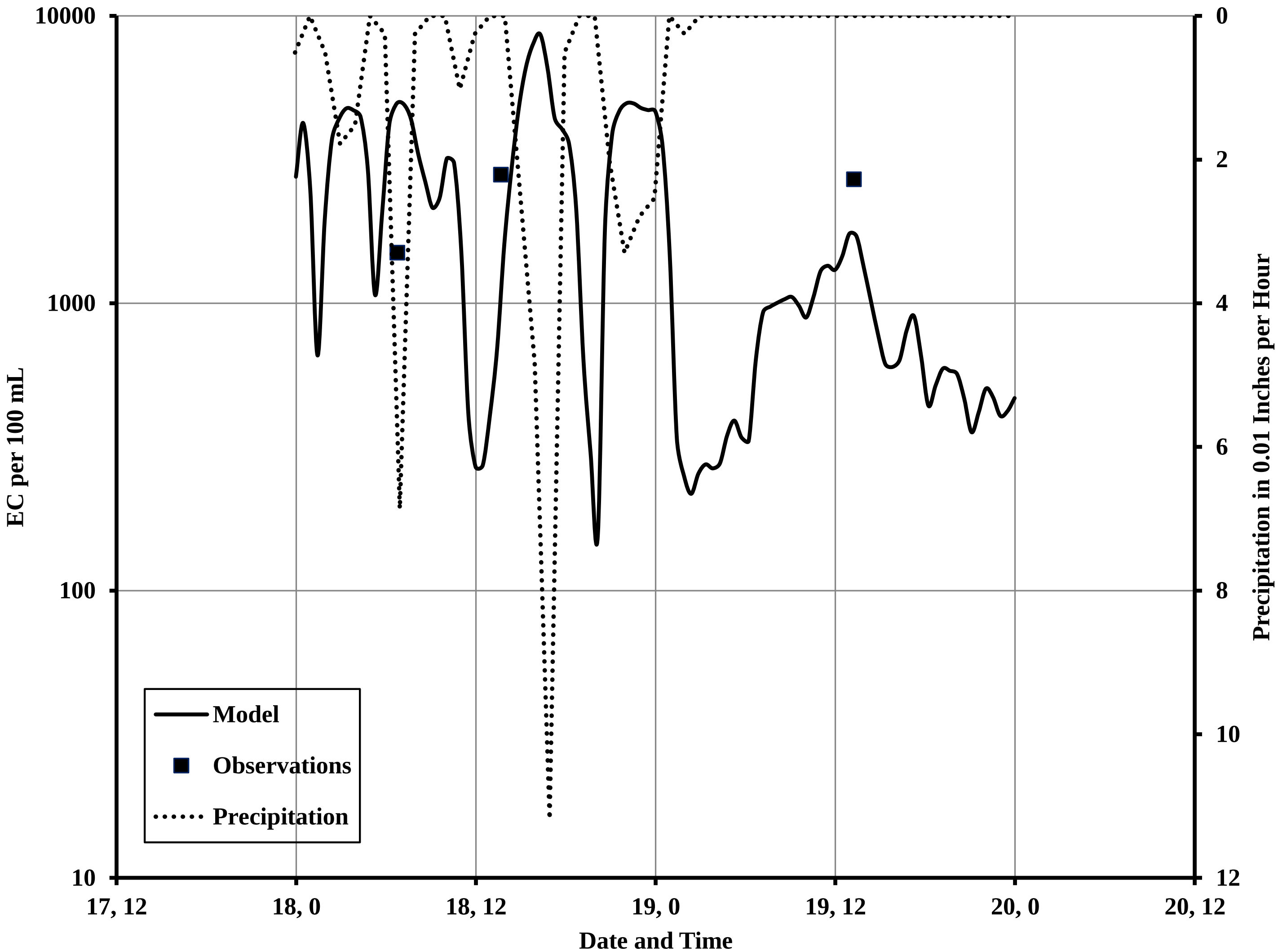

Figure VIII-10 - WAYNE WET EVENT 4&5 WITH PRECIPITATION MAY 18-19, 2011

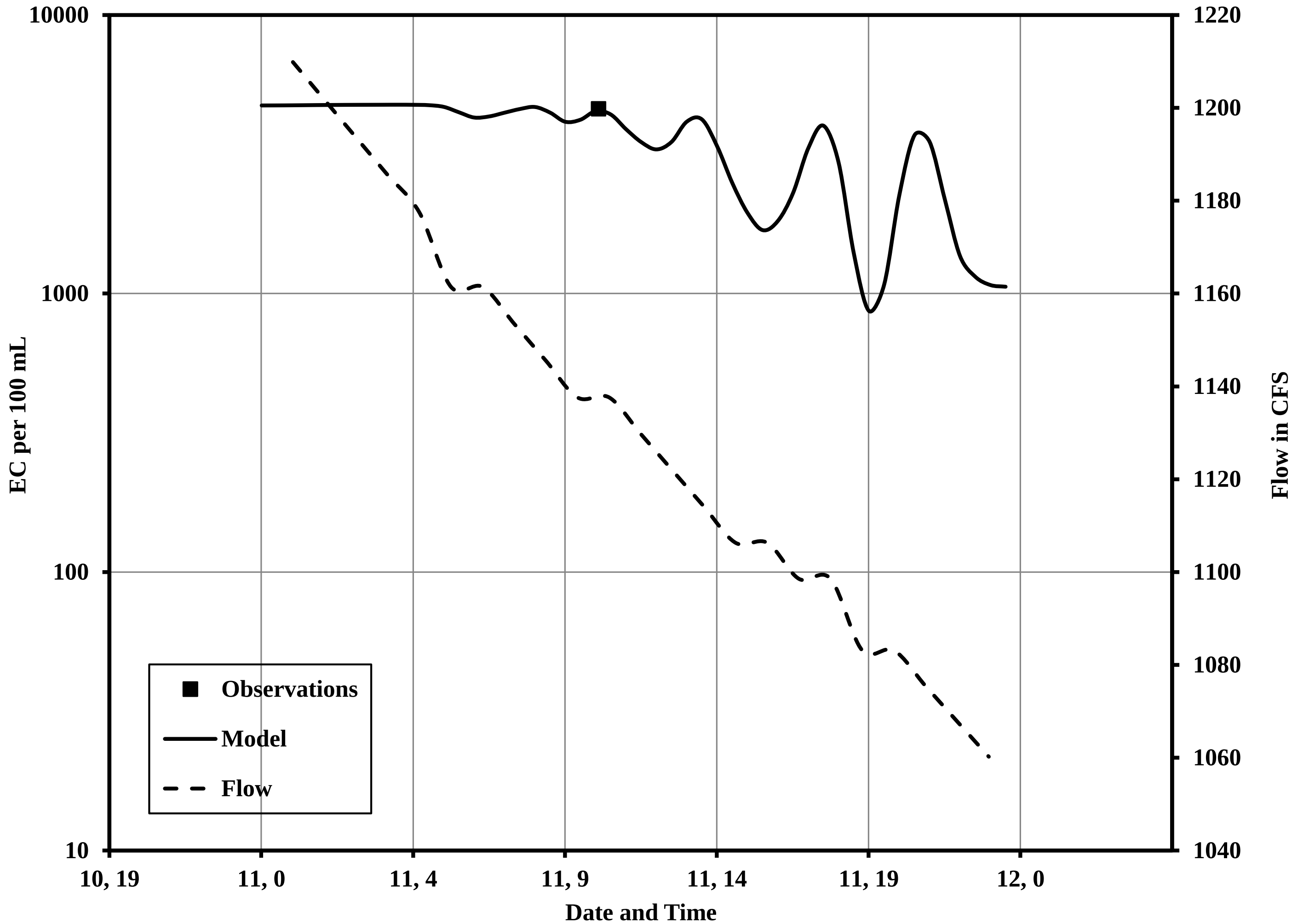

Figure VIII-11 - WAYNE WET EVENT 6 WITH FLOW OCTOBER 11, 2011

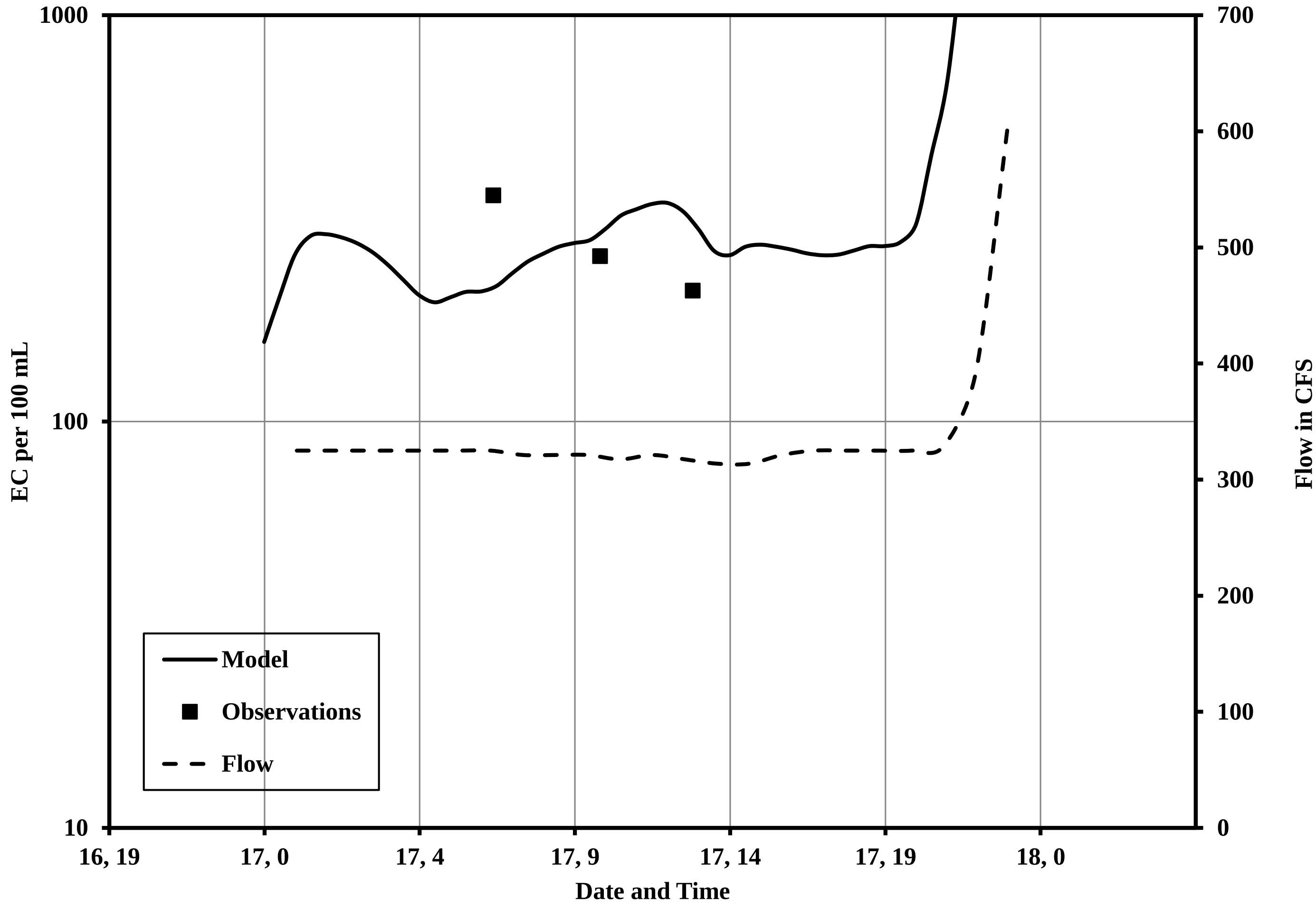

Figure VIII-12 - NORTHWEST DRY EVENT 1 JULY 17, 2009

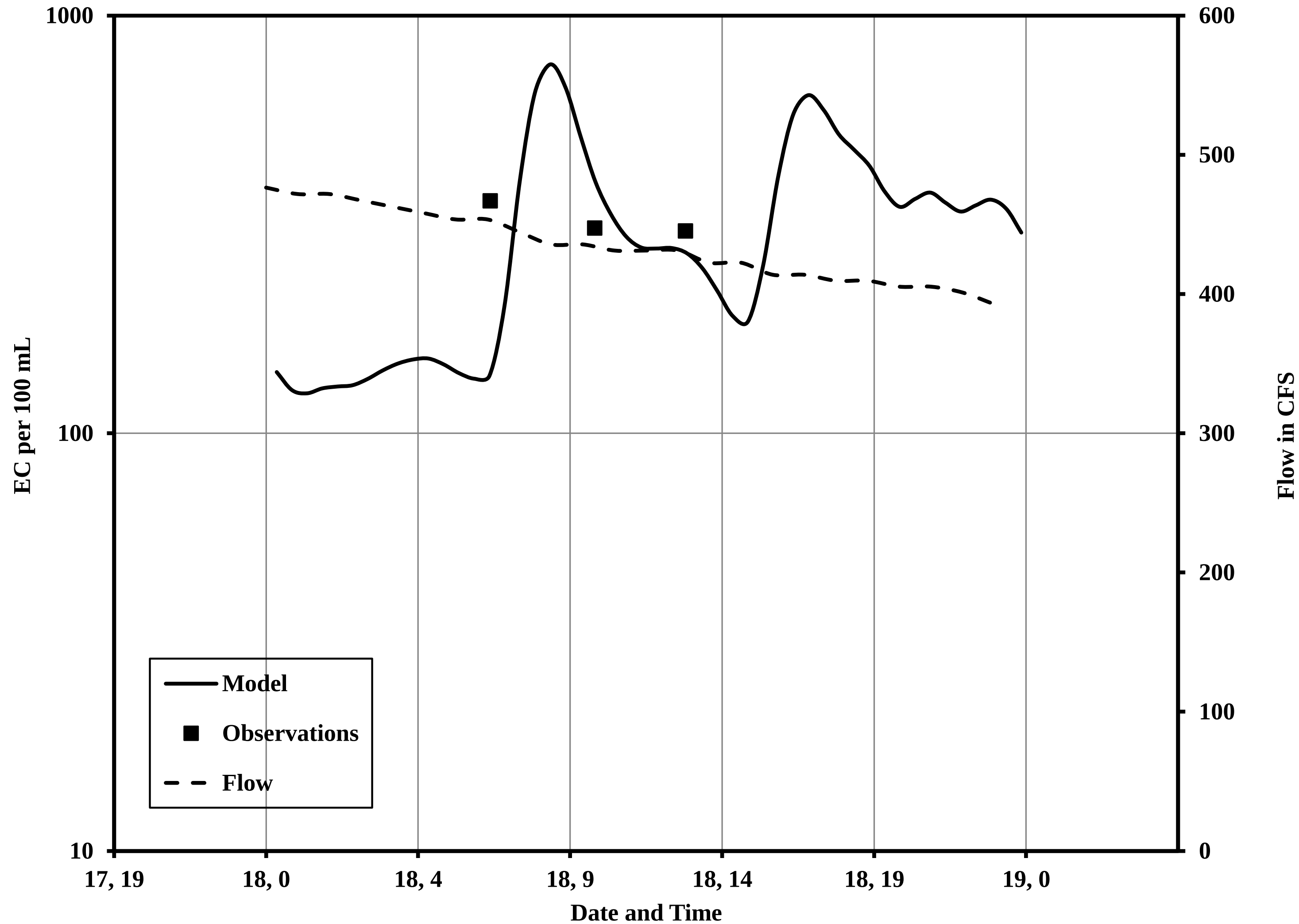

Figure VIII-13 - NORTHWEST DRY EVENT 2 AUGUST 18, 2009

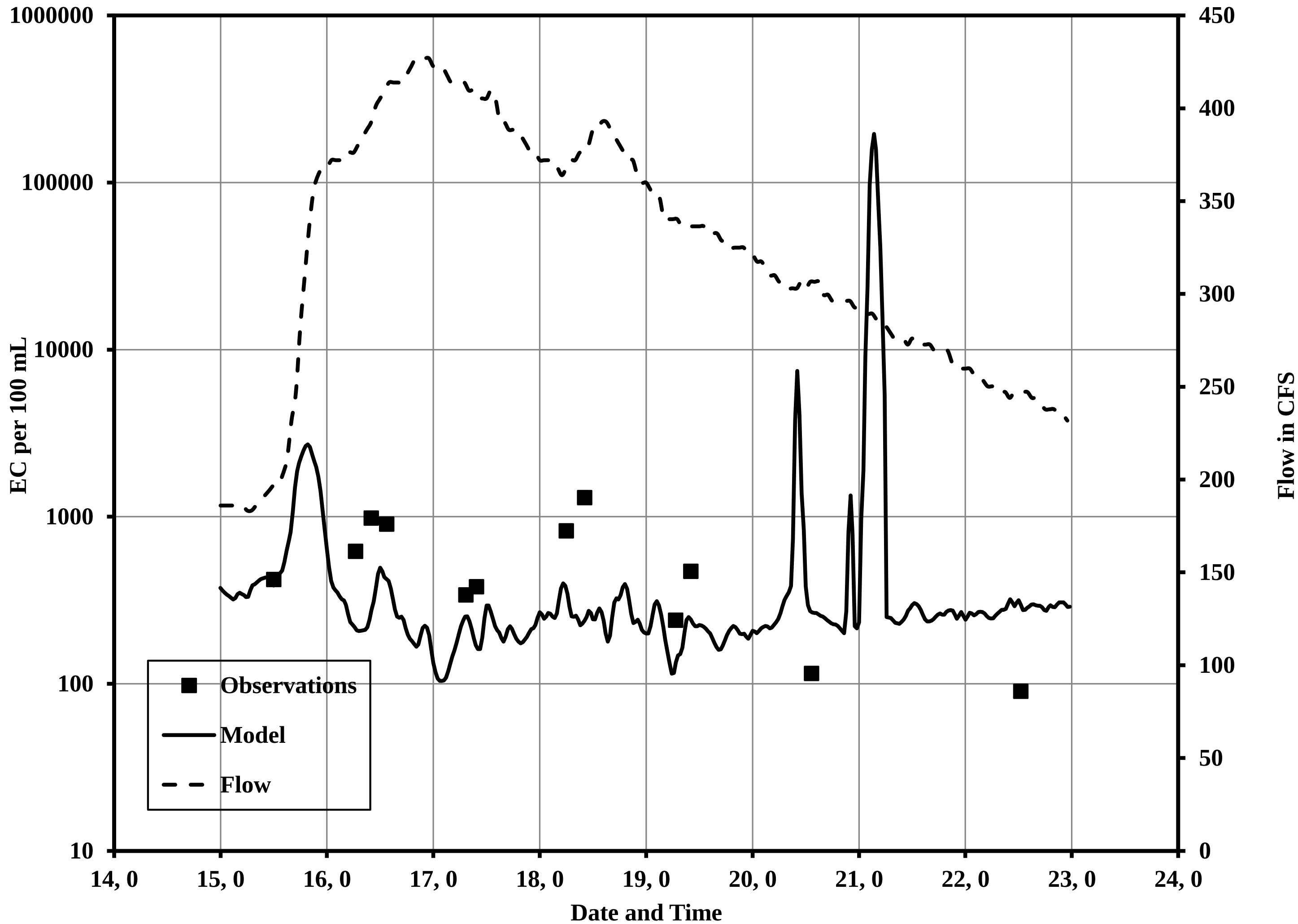

Figure VIII-14 - NORTHWEST WET EVENT 1 WITH FLOW OCTOBER 15-22, 2009

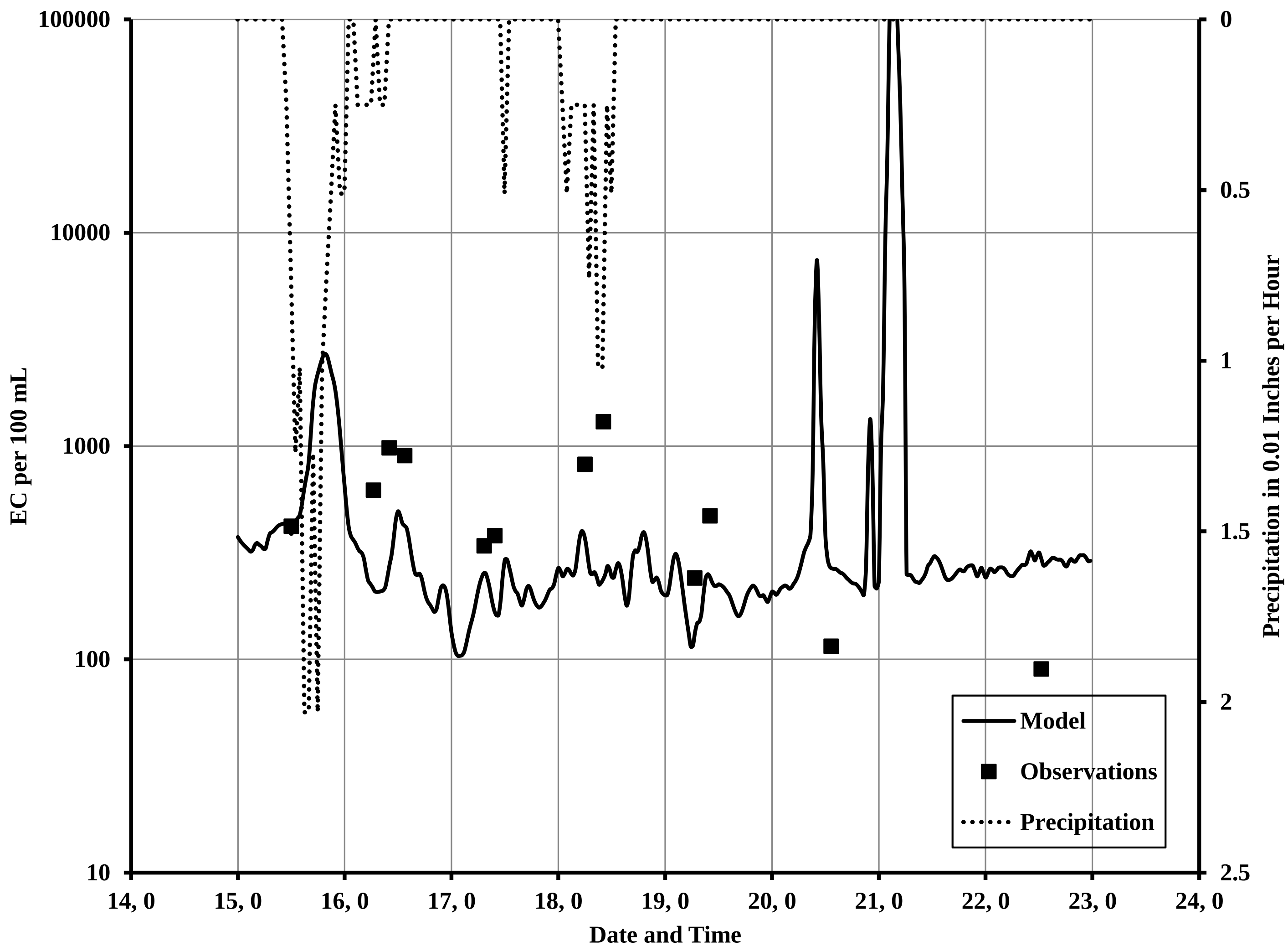

Figure VIII-15 - NORTHWEST WET EVENT 1 WITH PRECIPITATION OCTOBER 15-22, 2009

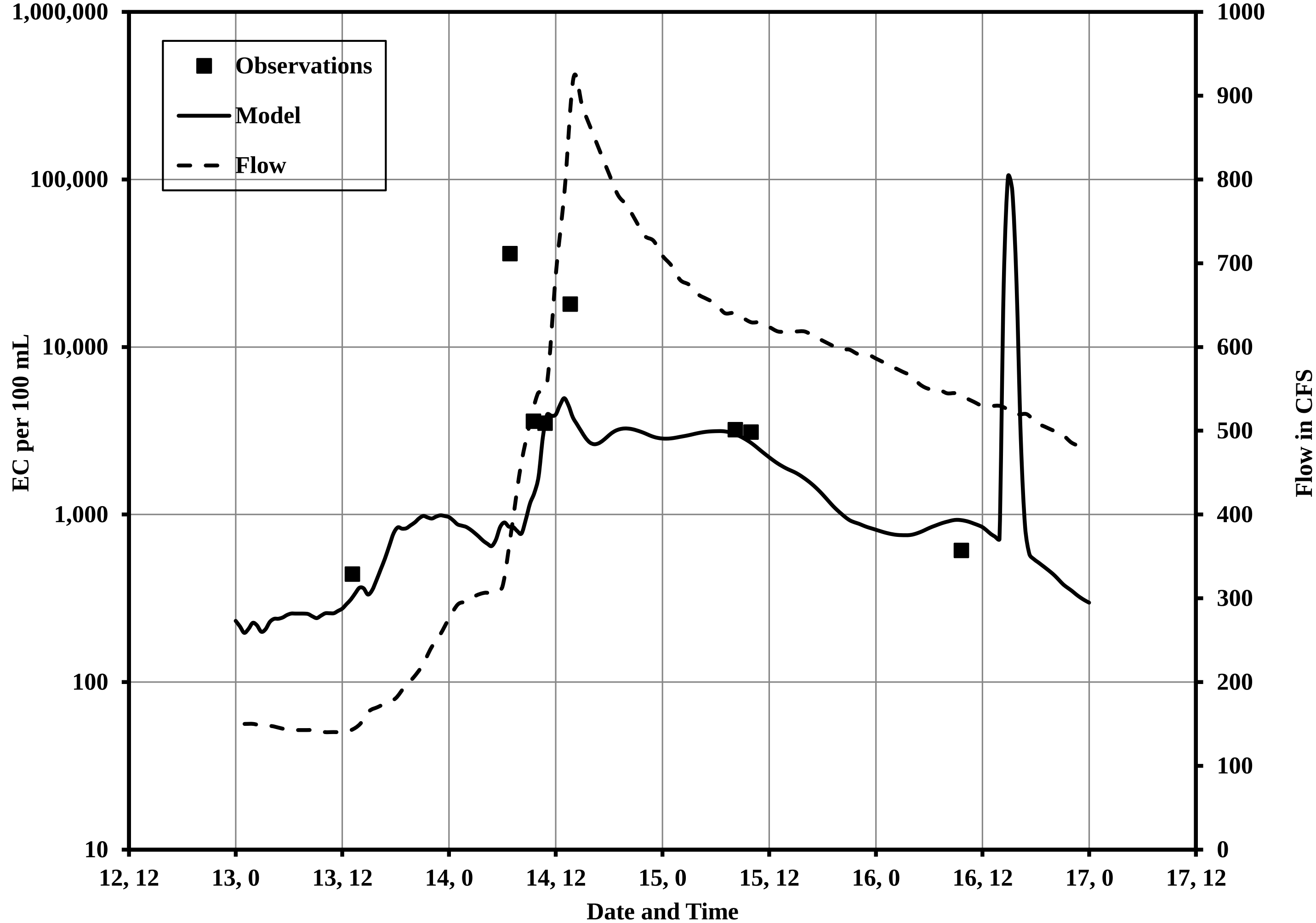

Figure VIII-16 - NORTHWEST WET EVENT 2 WITH FLOW JULY 13-16, 2010

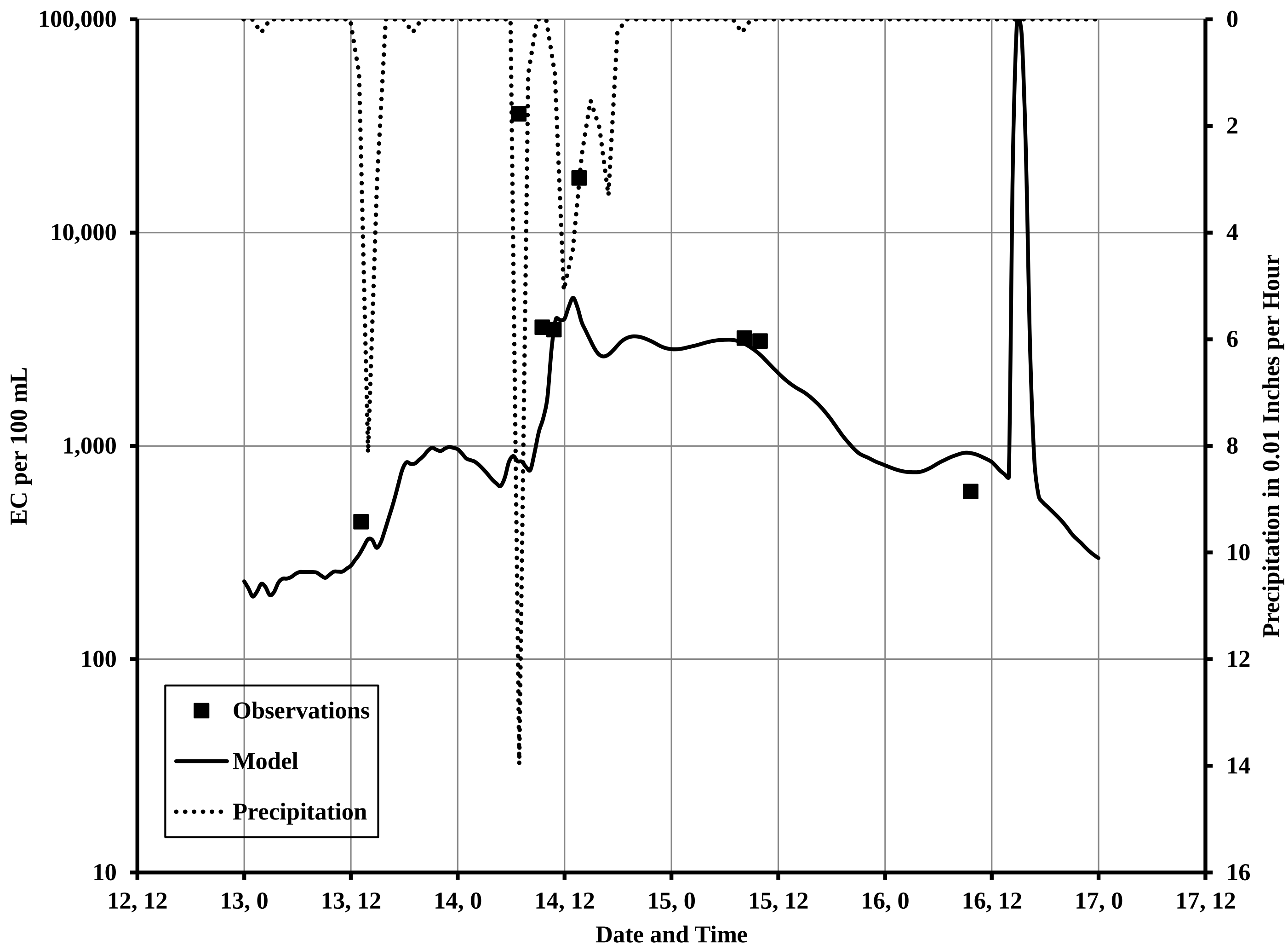

Figure VIII-17 - NORTHWEST WET EVENT 2 WITH PRECIPITATION JULY 13-16, 2010

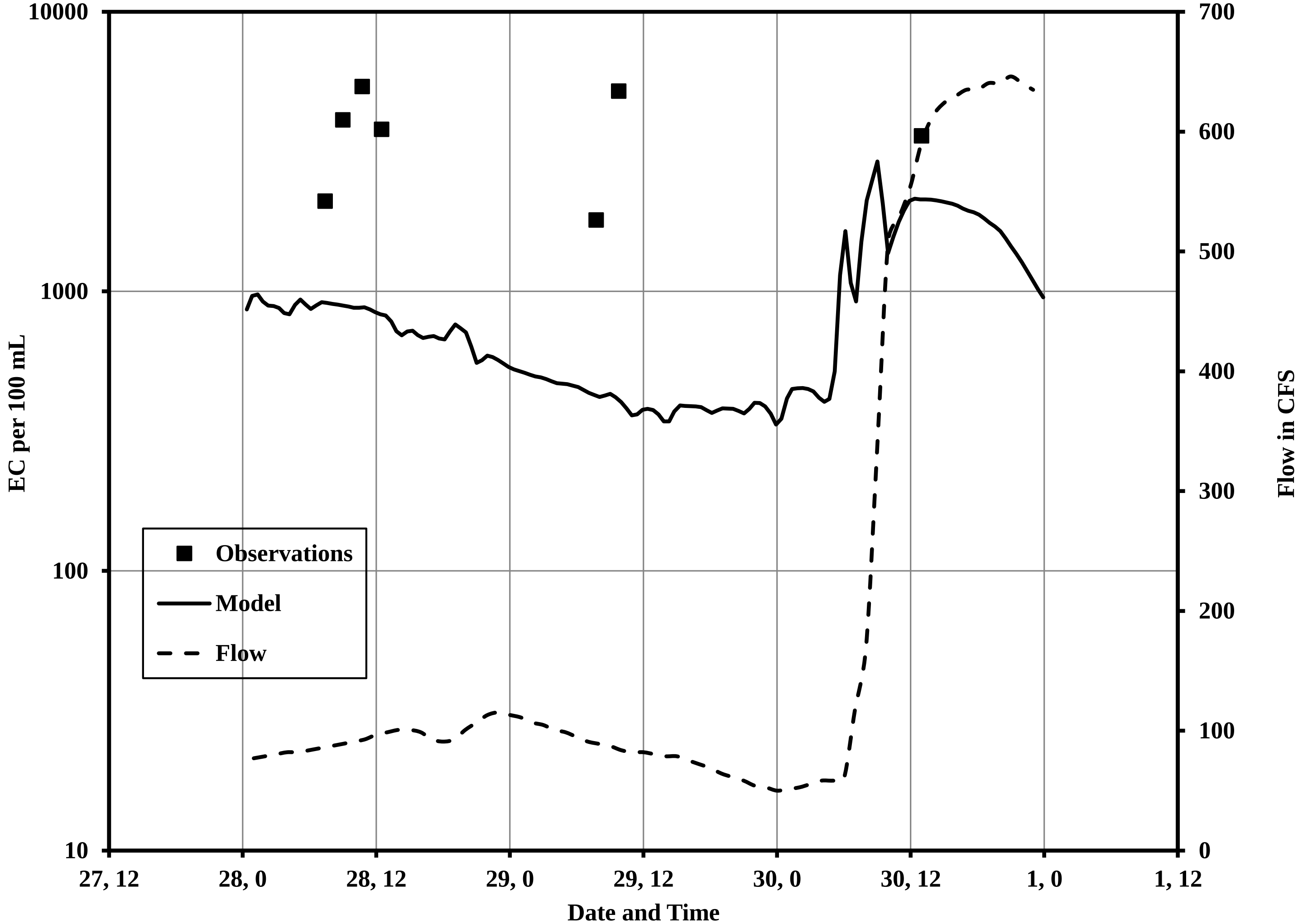

Figure VIII-18 - NORTHWEST WET EVENT 3 WITH FLOW OCTOBER 15, 28-30, 2010

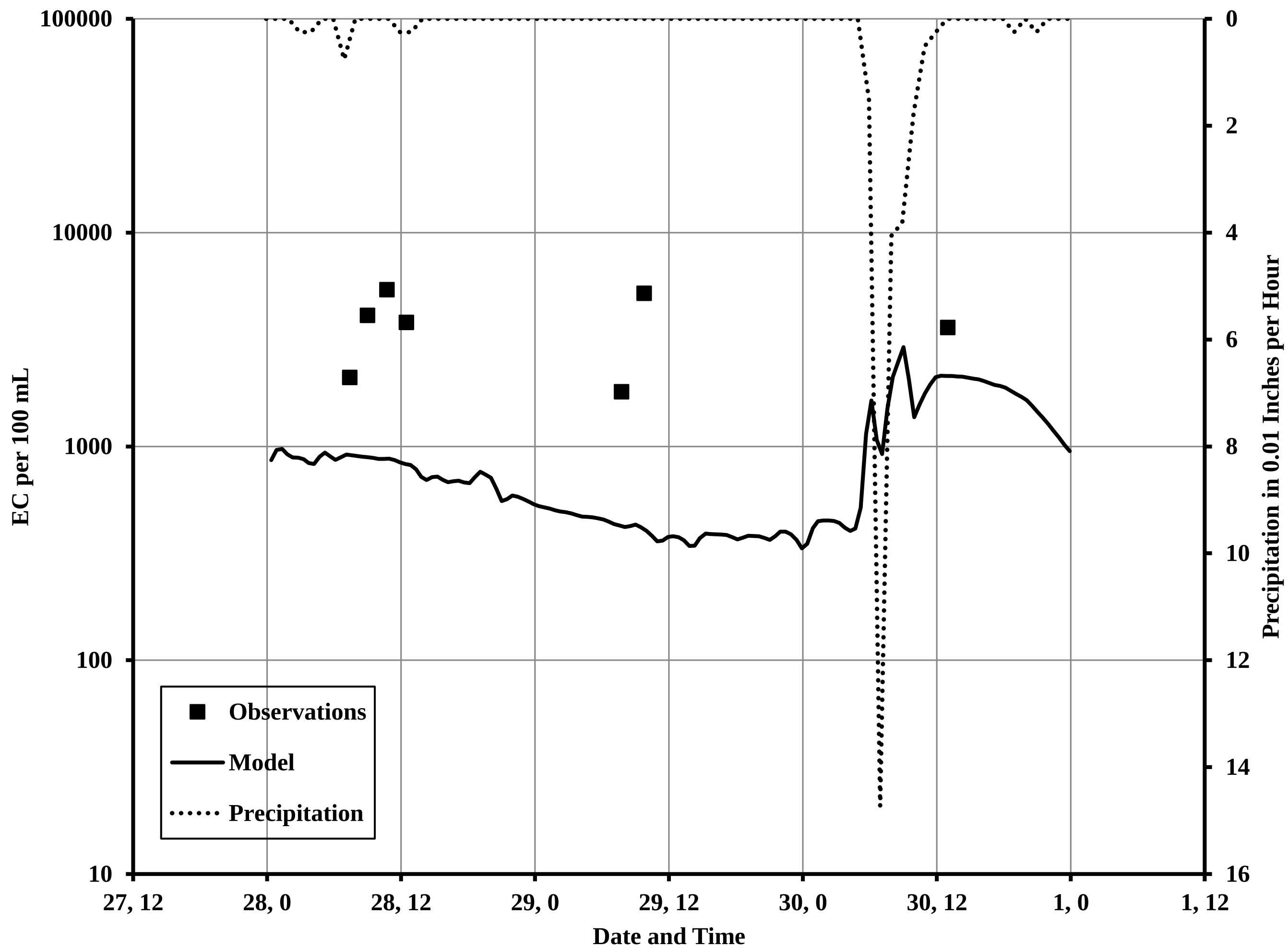

Figure VIII-19 - NORTHWEST WET EVENT 3 WITH PRECIPITATION OCTOBER 15, 28-30, 2010

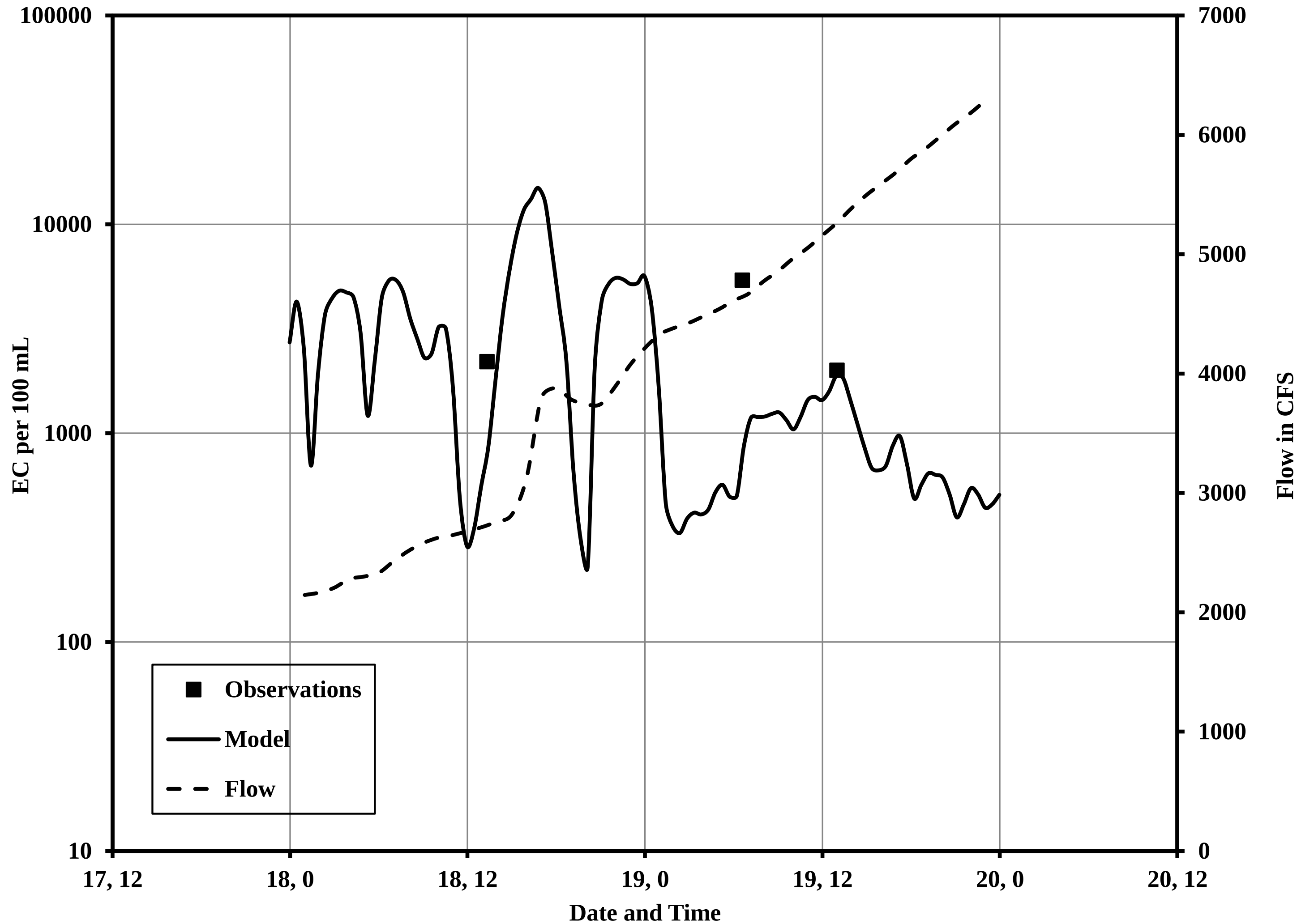

Figure VIII-20 - NORTHWEST WET EVENT 4&5 WITH FLOW MAY 18-19, 2011

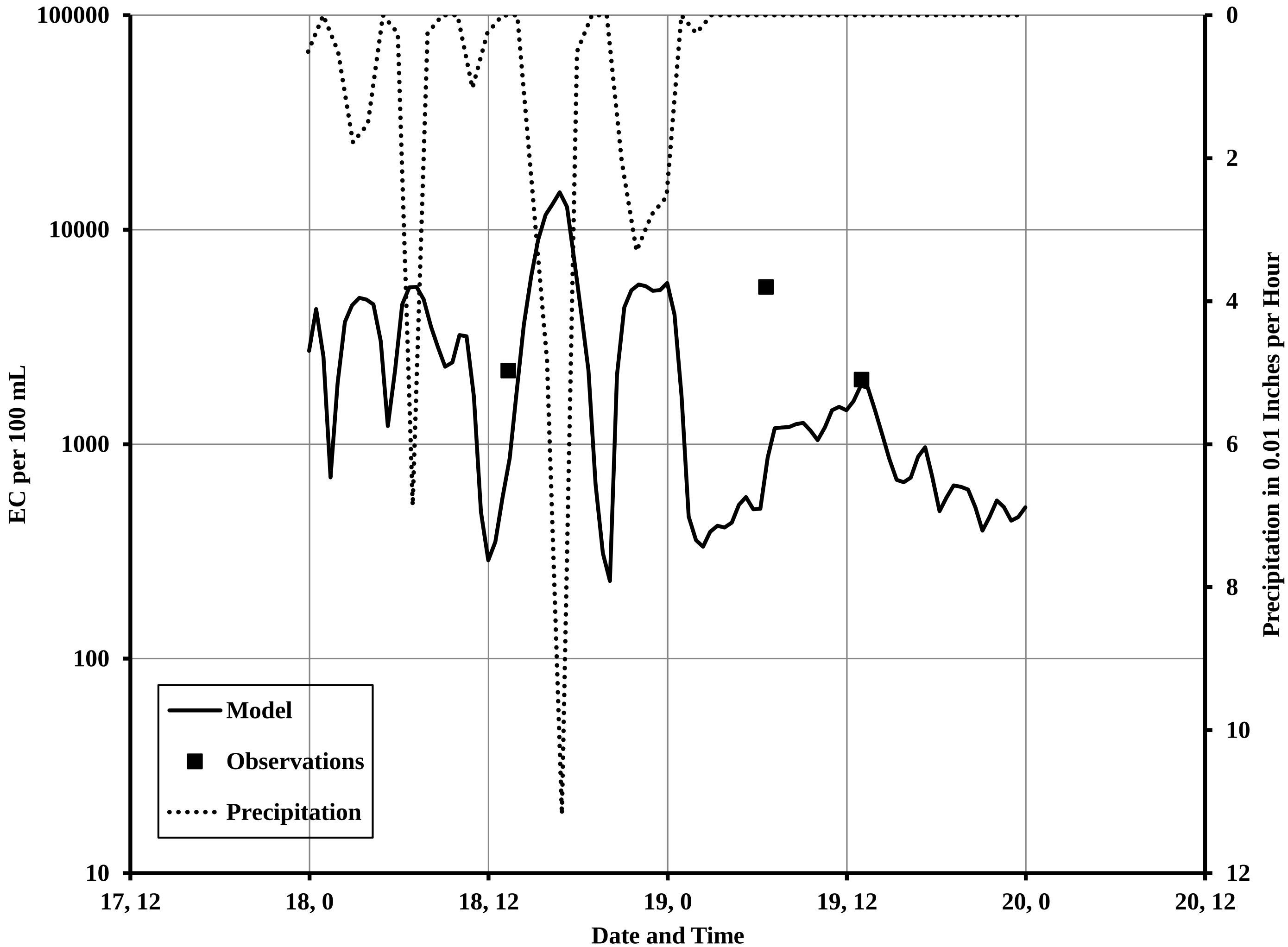

Figure VIII-21 – NORTHWEST WET EVENT 4&5 WITH PRECIPITATION MAY 18-19, 2011

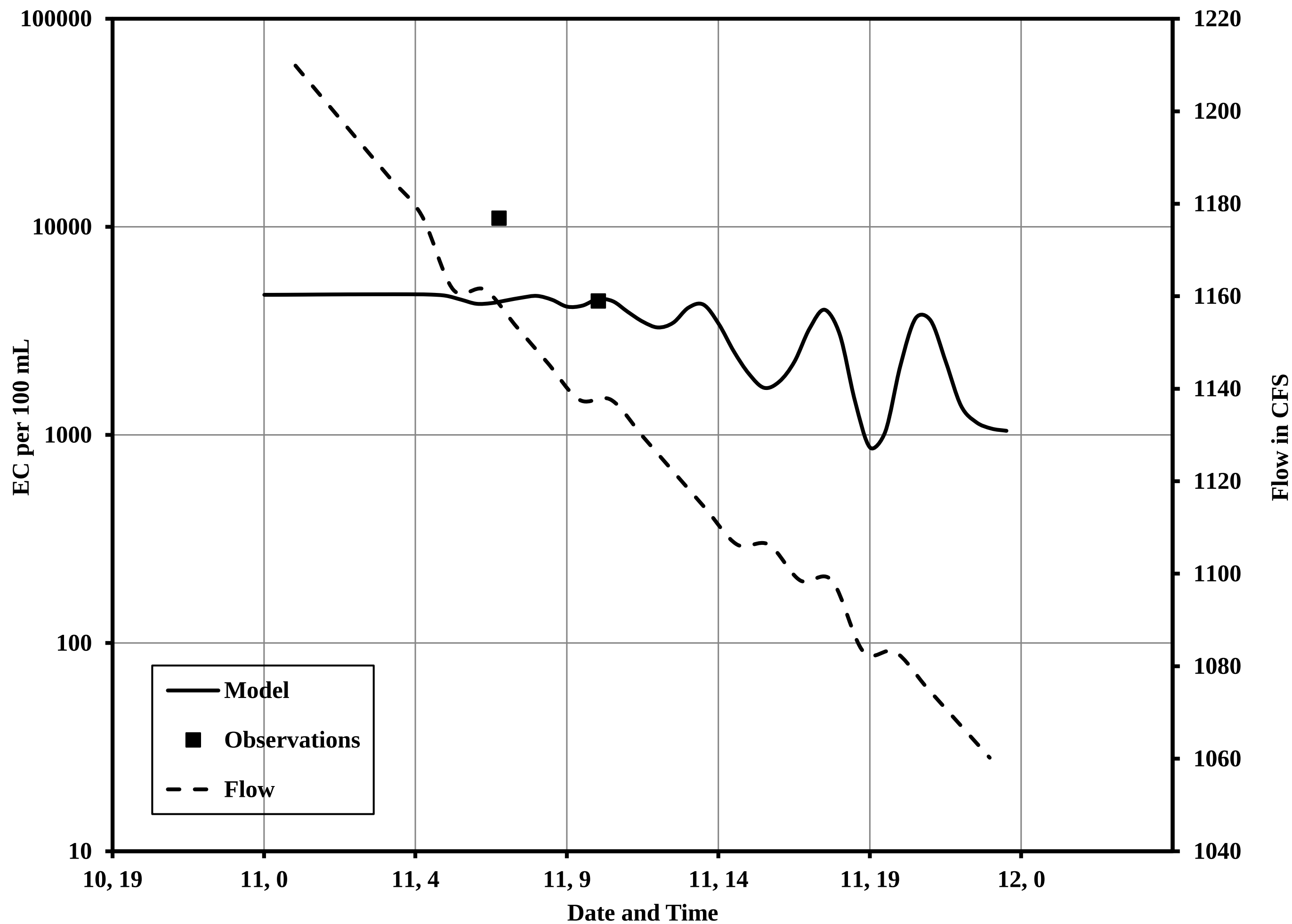

Figure VIII-22 – NORTHWEST WET EVENT 6 WITH FLOW OCTOBER 11, 2011

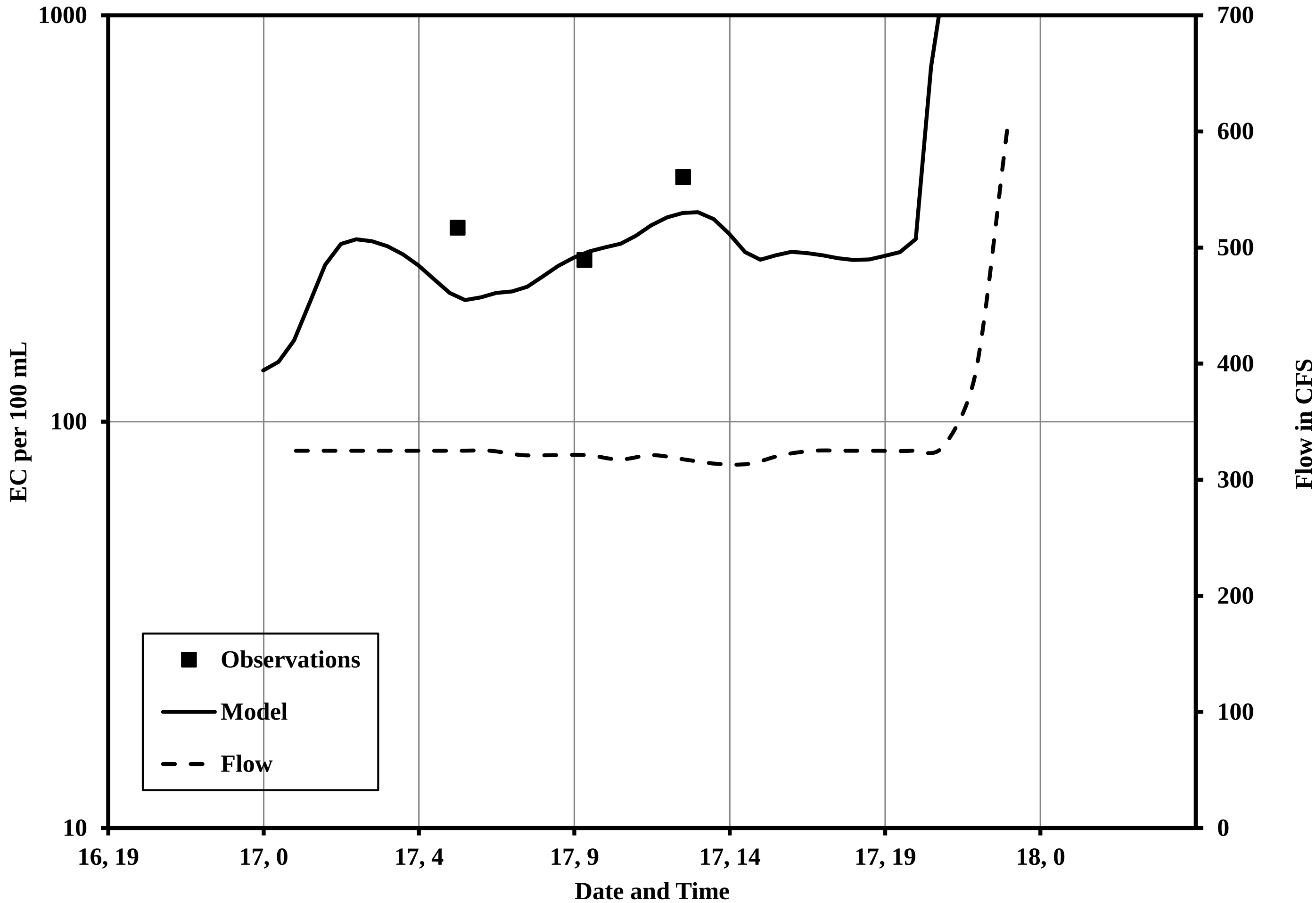

Figure VIII-23 - LINCOLN DRY EVENT 1 JULY 17, 2009

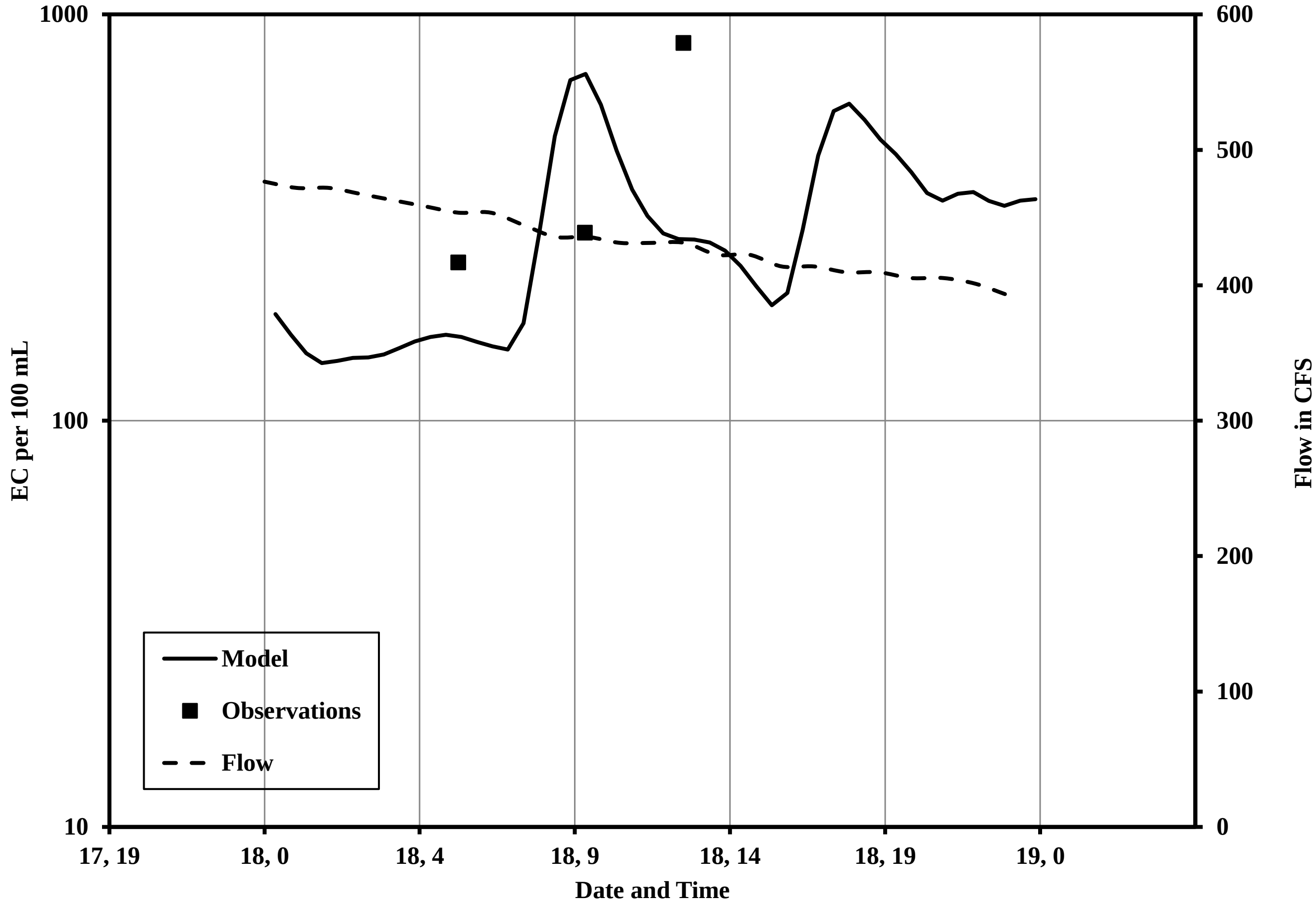

Figure VIII-24 - LINCOLN DRY EVENT 2 AUGUST 18, 2009

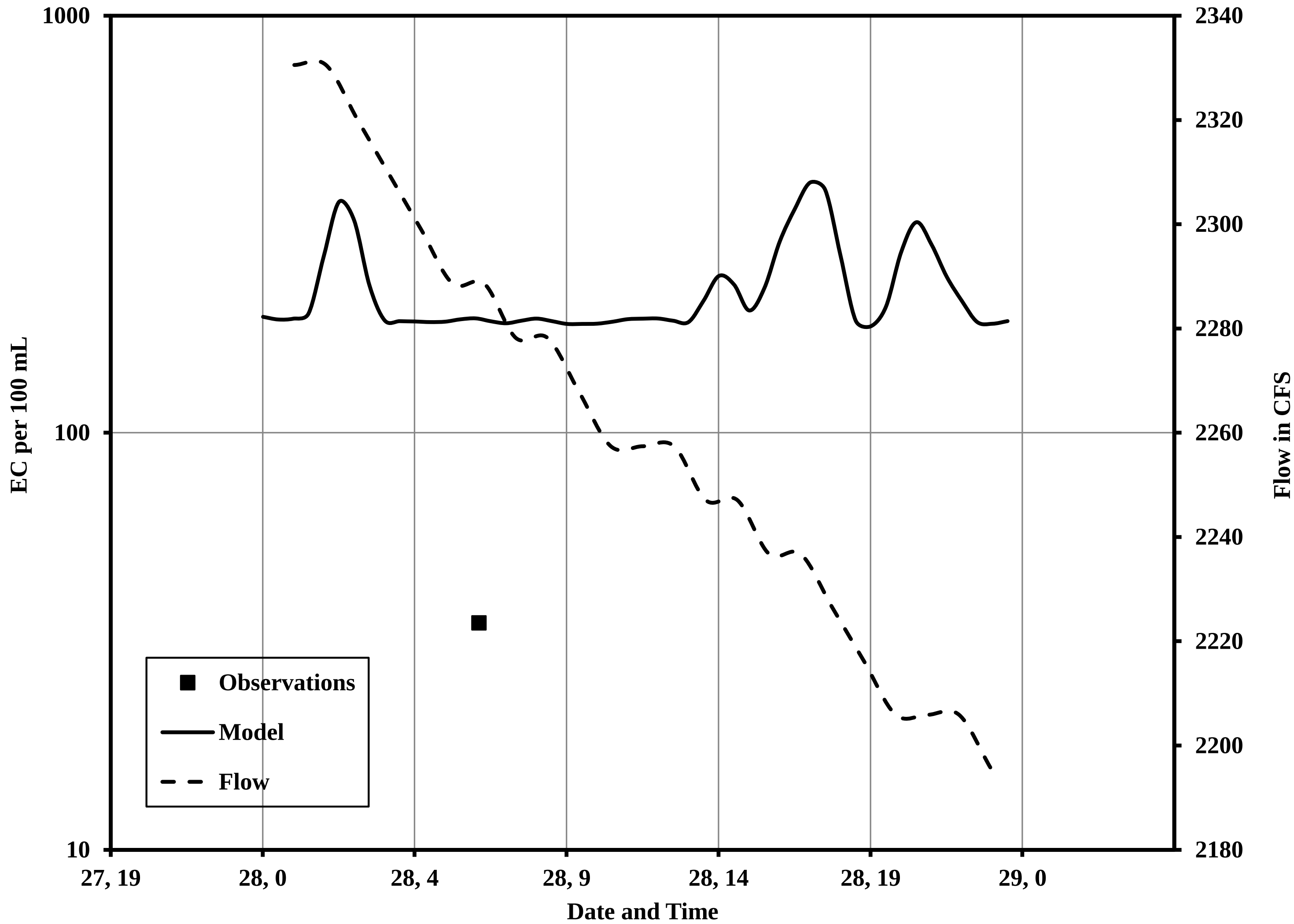

Figure VIII-25 - LINCOLN DRY EVENT 3 MARCH 28, 2011

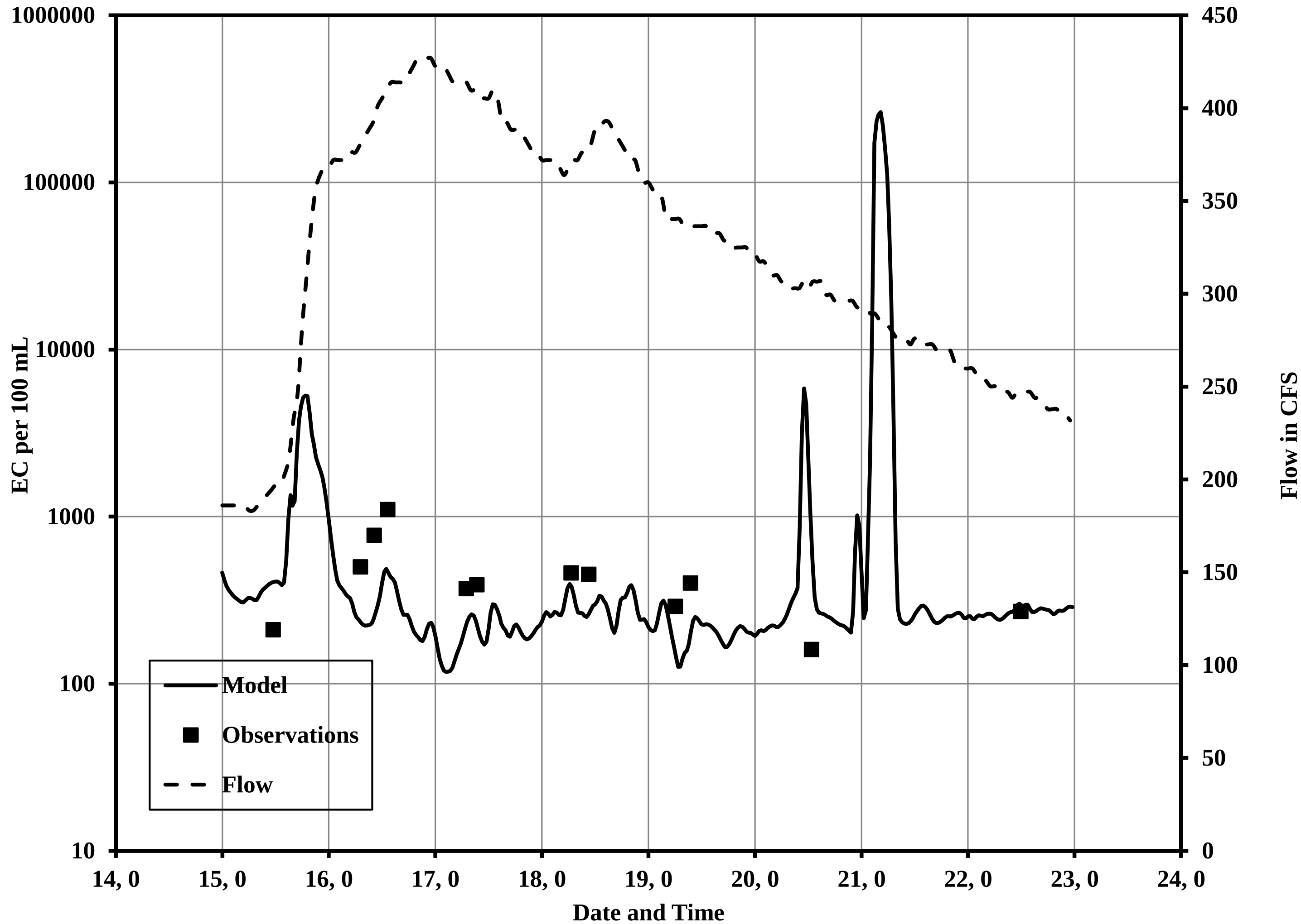

Figure VIII-26 - LINCOLN WET EVENT 1 WITH FLOW OCTOBER 15-22, 2009

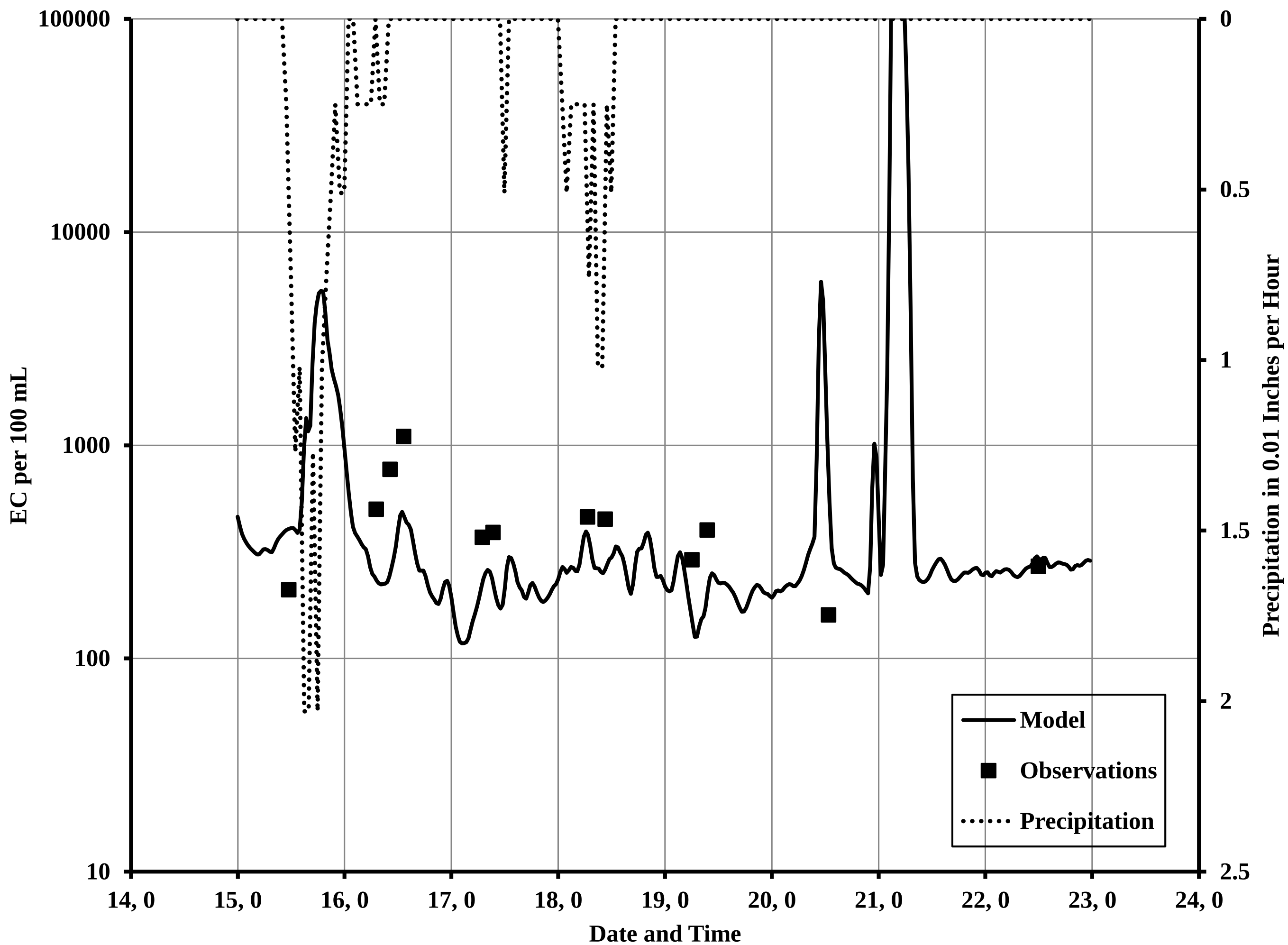

Figure VIII-27 - LINCOLN WET EVENT 1 WITH PRECIPITATION OCTOBER 15-22, 2009

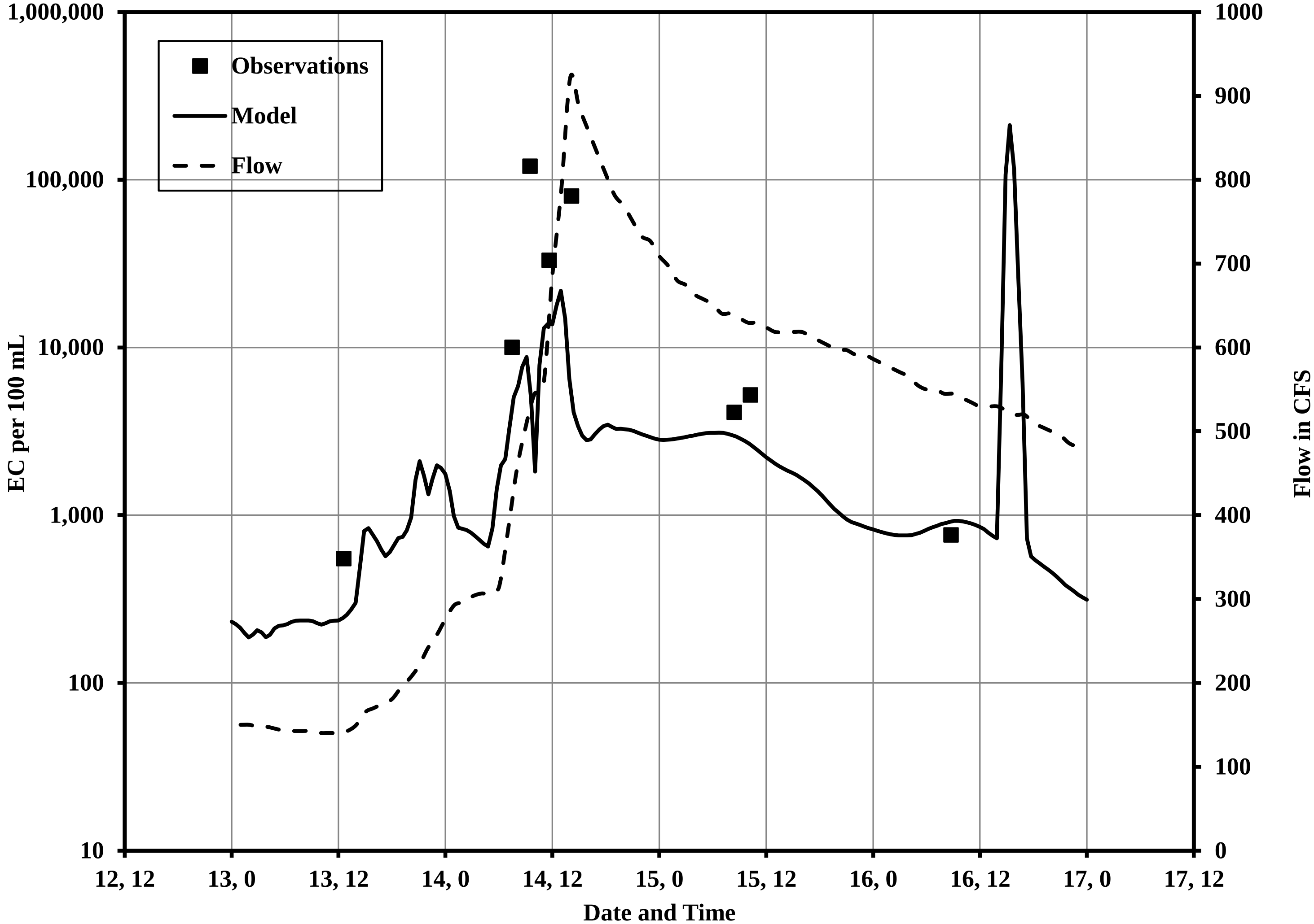

Figure VIII-28 - LINCOLN WET EVENT 2 WITH FLOW JULY 13-16, 2010

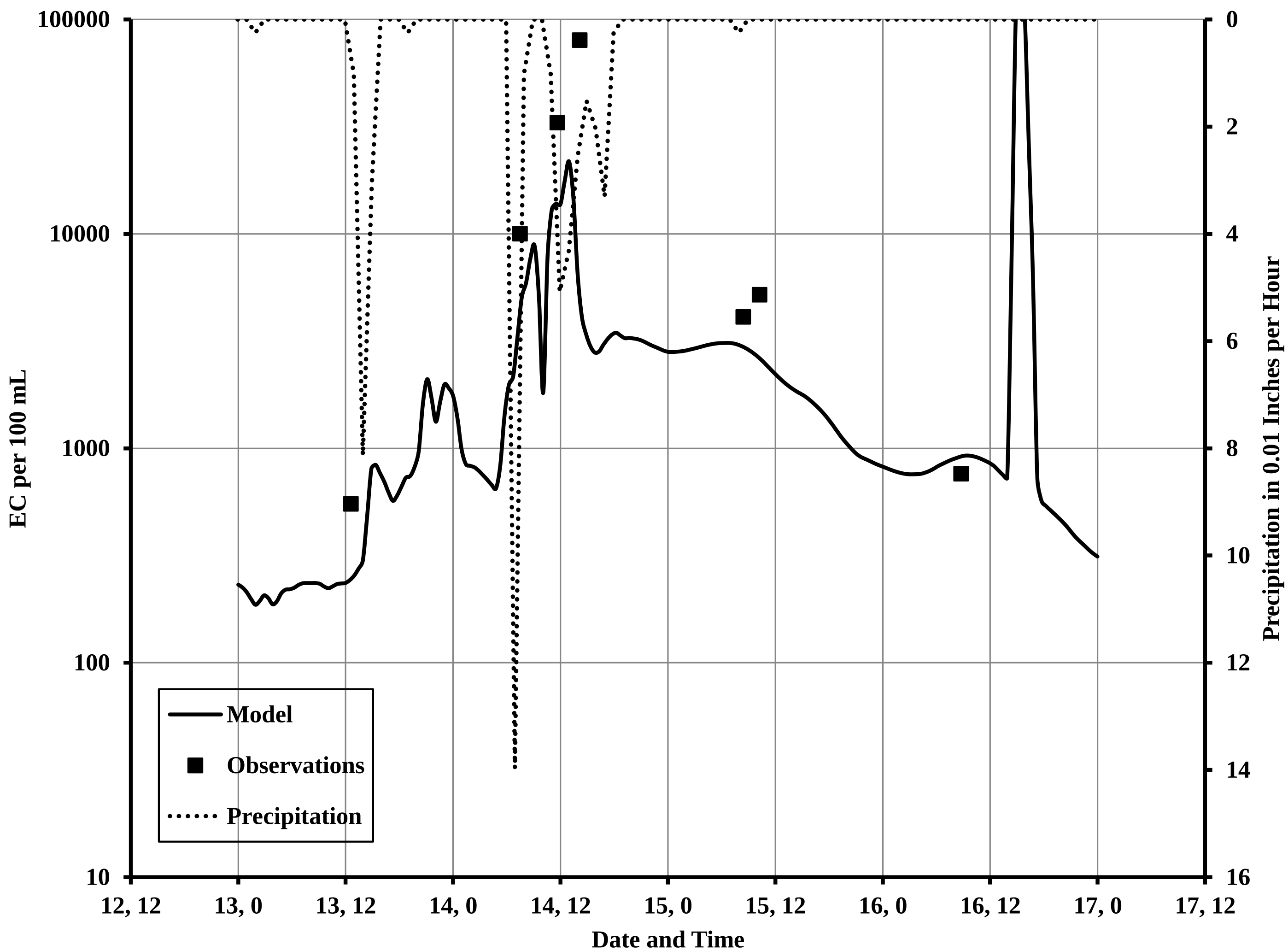

Figure VIII-29 - LINCOLN WET EVENT 2 WITH PRECIPITATION JULY 13-16, 2010

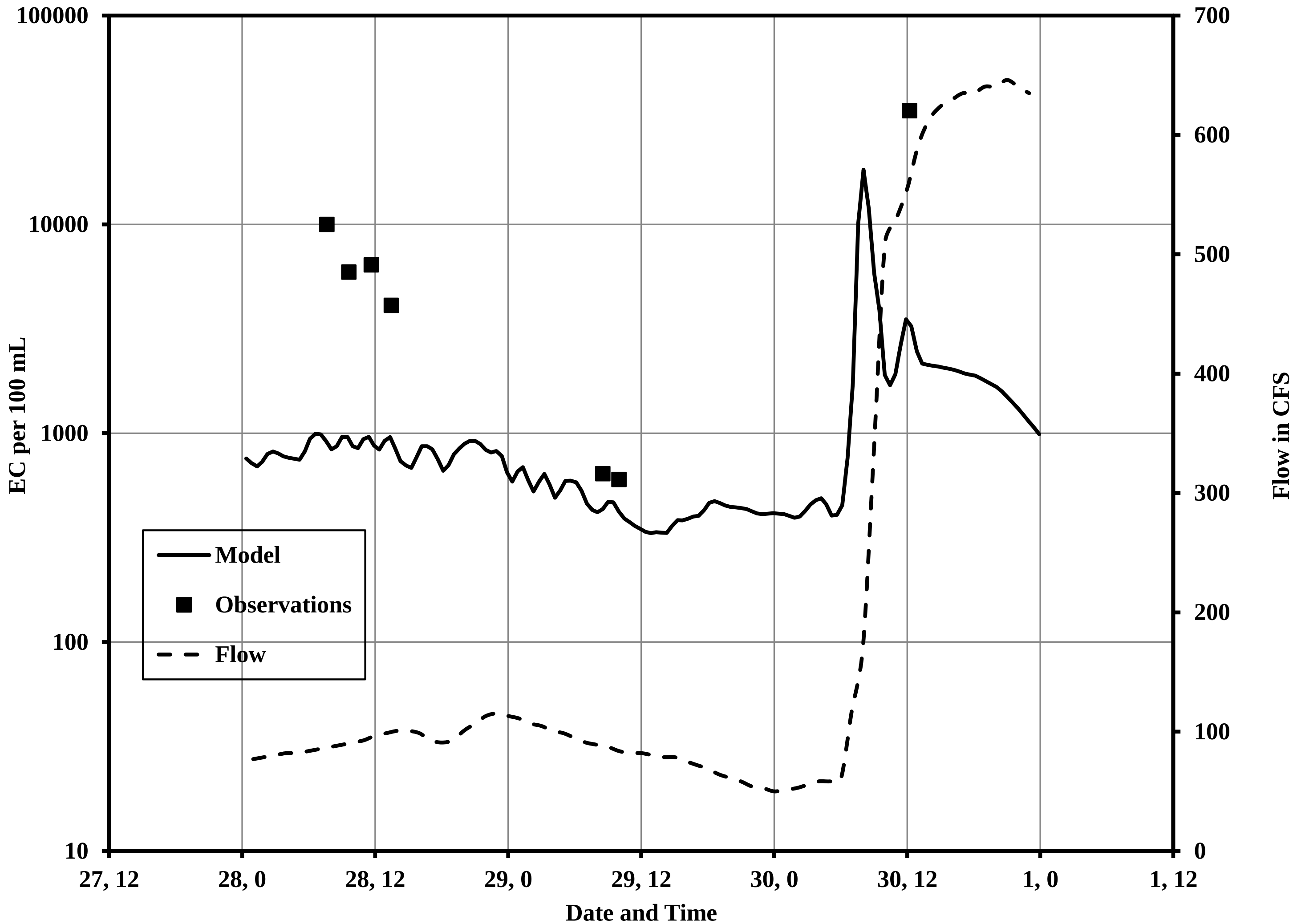

Figure VIII-30 - LINCOLN WET EVENT 3 WITH FLOW OCTOBER 15, 28-30, 2010

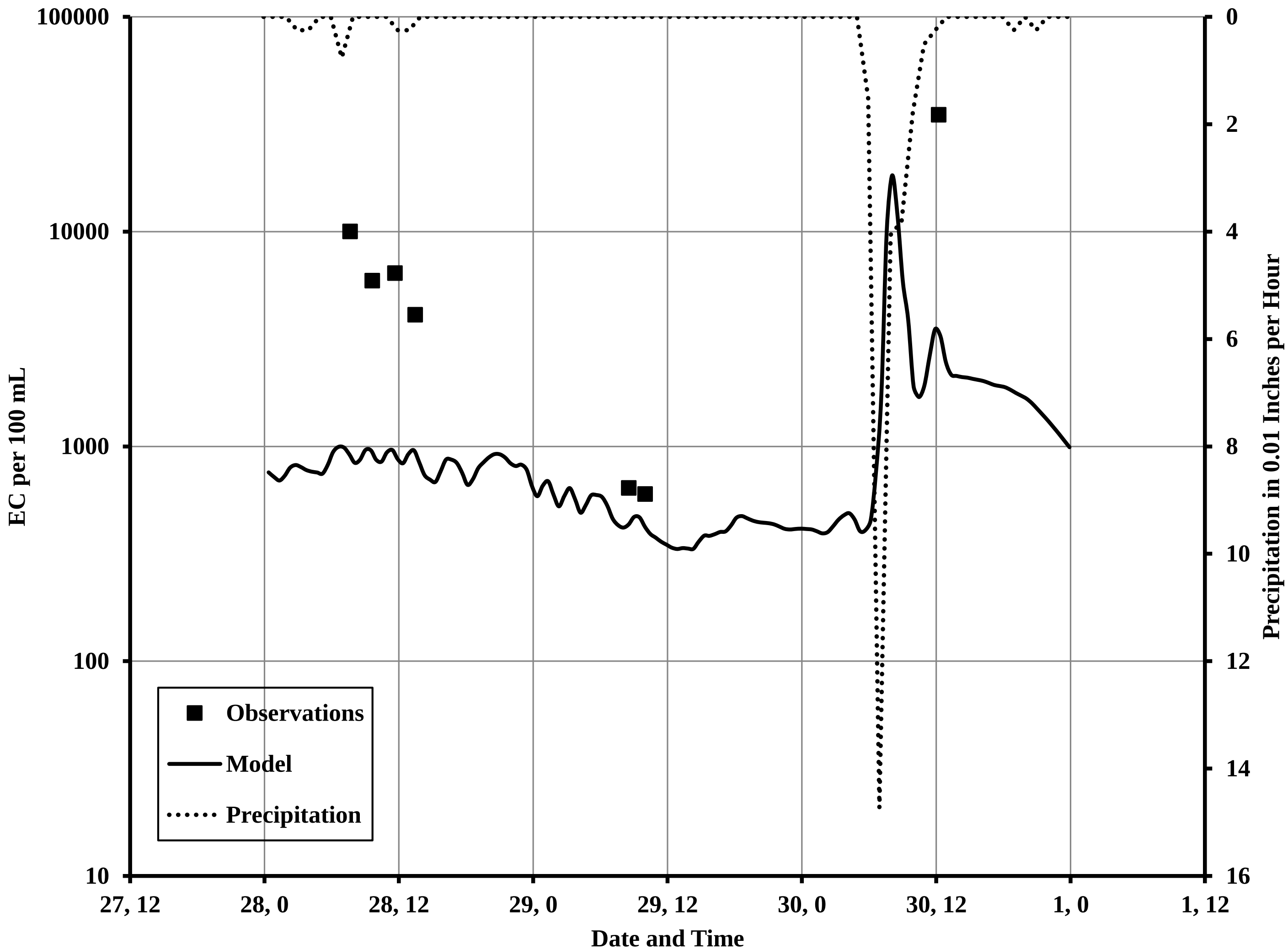

Figure VIII-31 - LINCOLN WET EVENT 3 WITH PRECIPITATION OCTOBER 15, 28-30, 2010

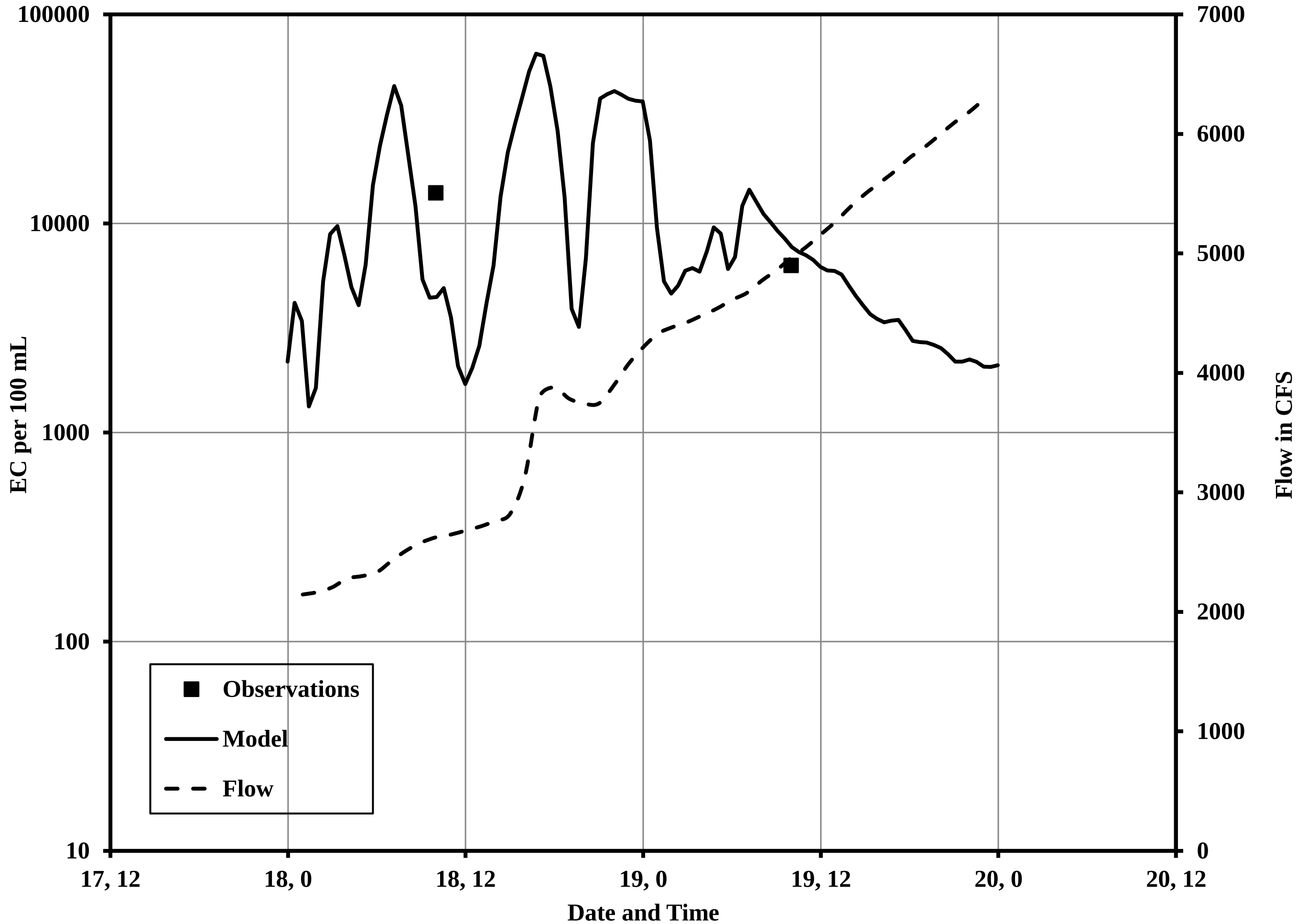

Figure VIII-32 - LINCOLN WET EVENT 4&5 WITH FLOW MAY 18-19, 2011

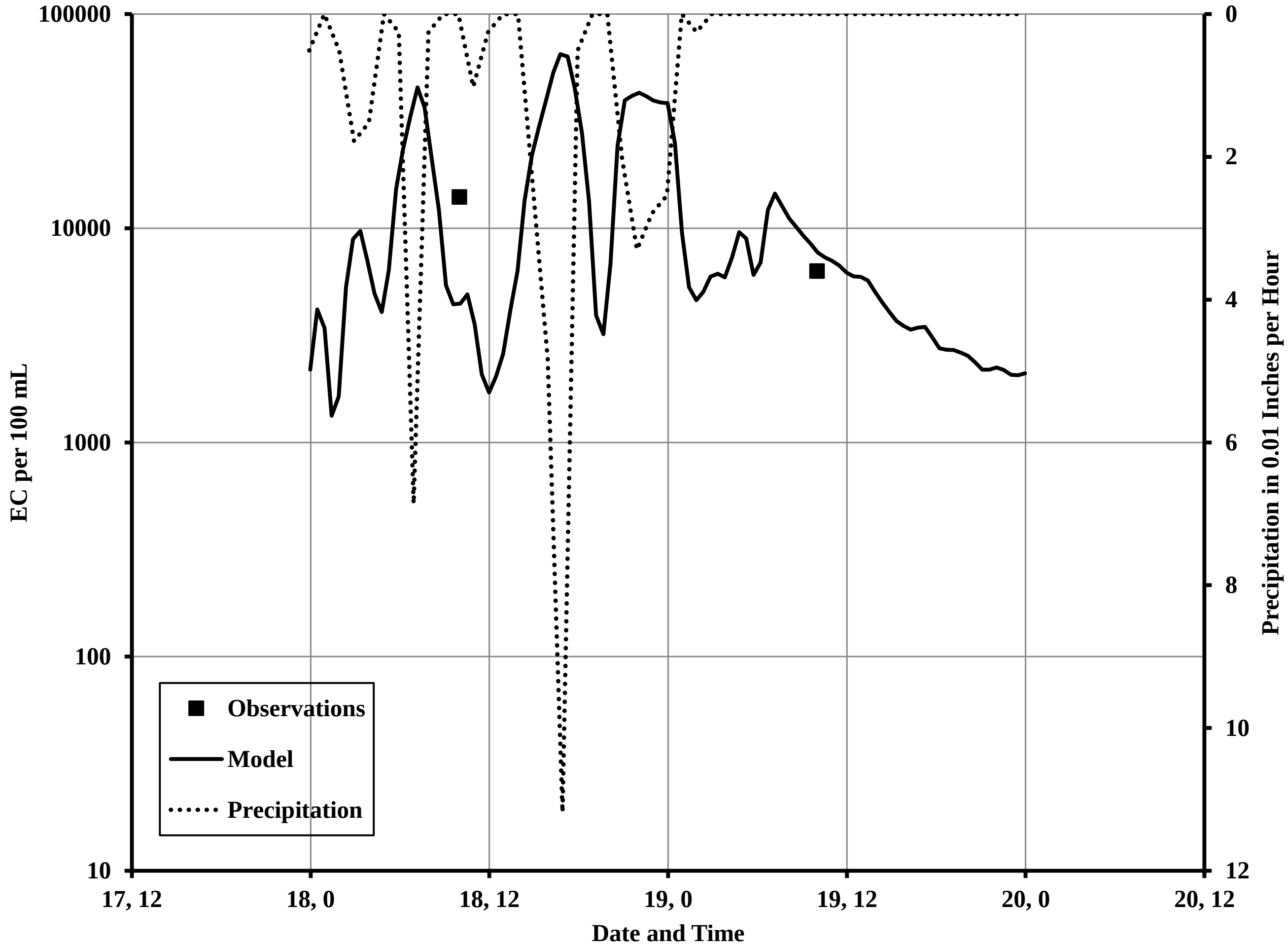

Figure VIII-33 - LINCOLN WET EVENT 4&5 WITH PRECIPITATION MAY 18-19, 2011

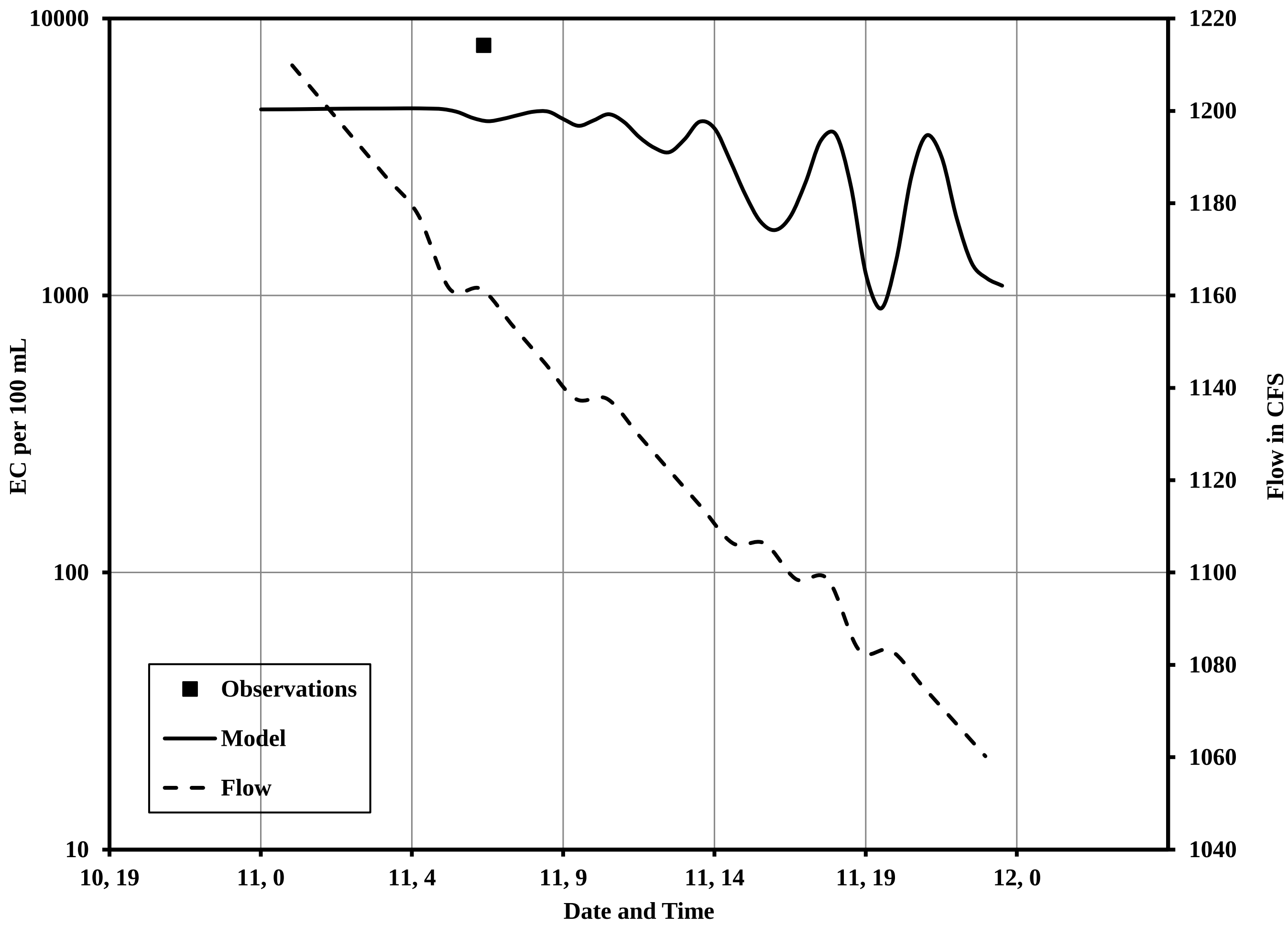

Figure VIII-34 - LINCOLN WET EVENT 6 WITH FLOW OCTOBER 11, 2011

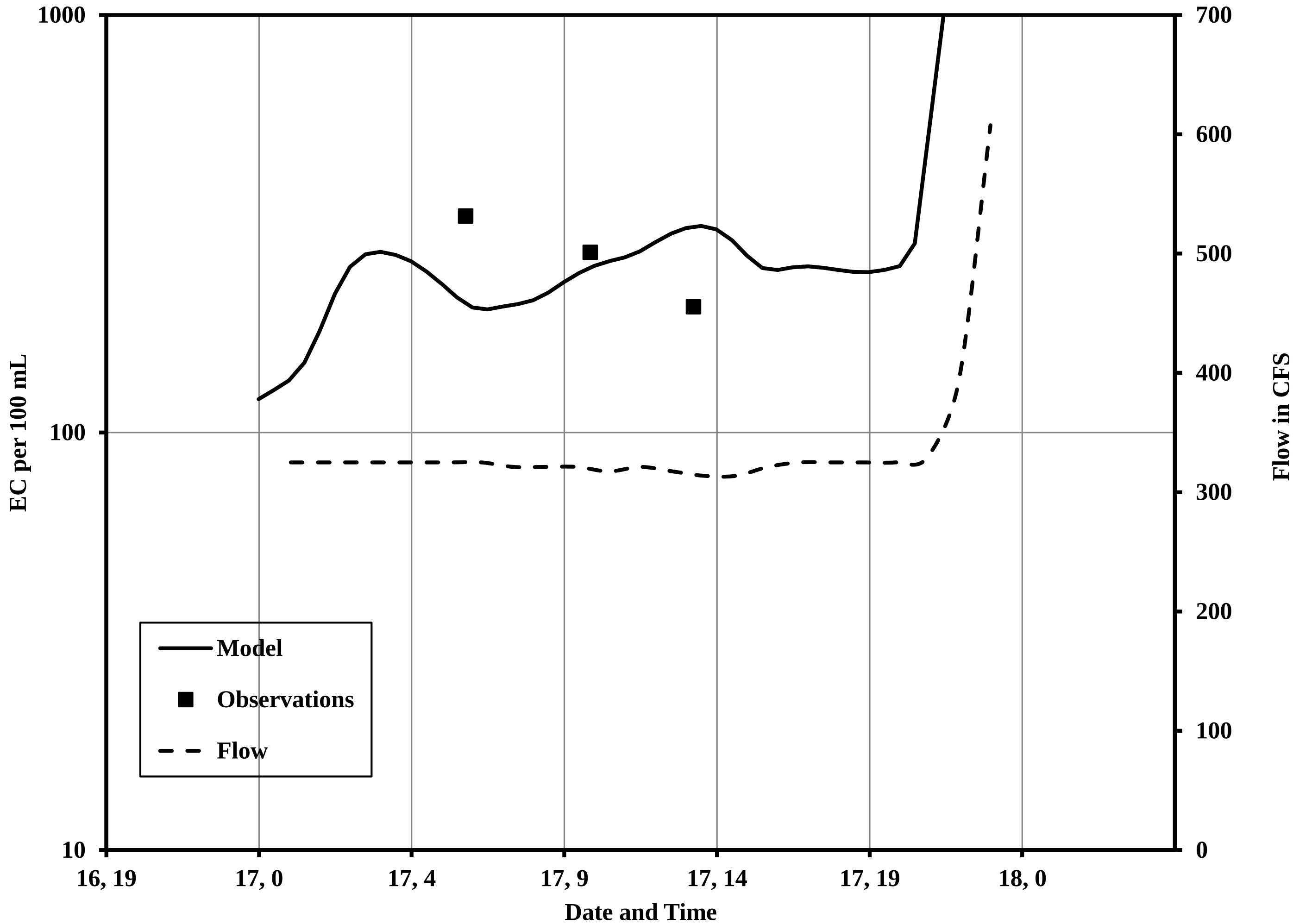

Figure VIII-35 - MORLOT DRY EVENT 1 JULY 17, 2009

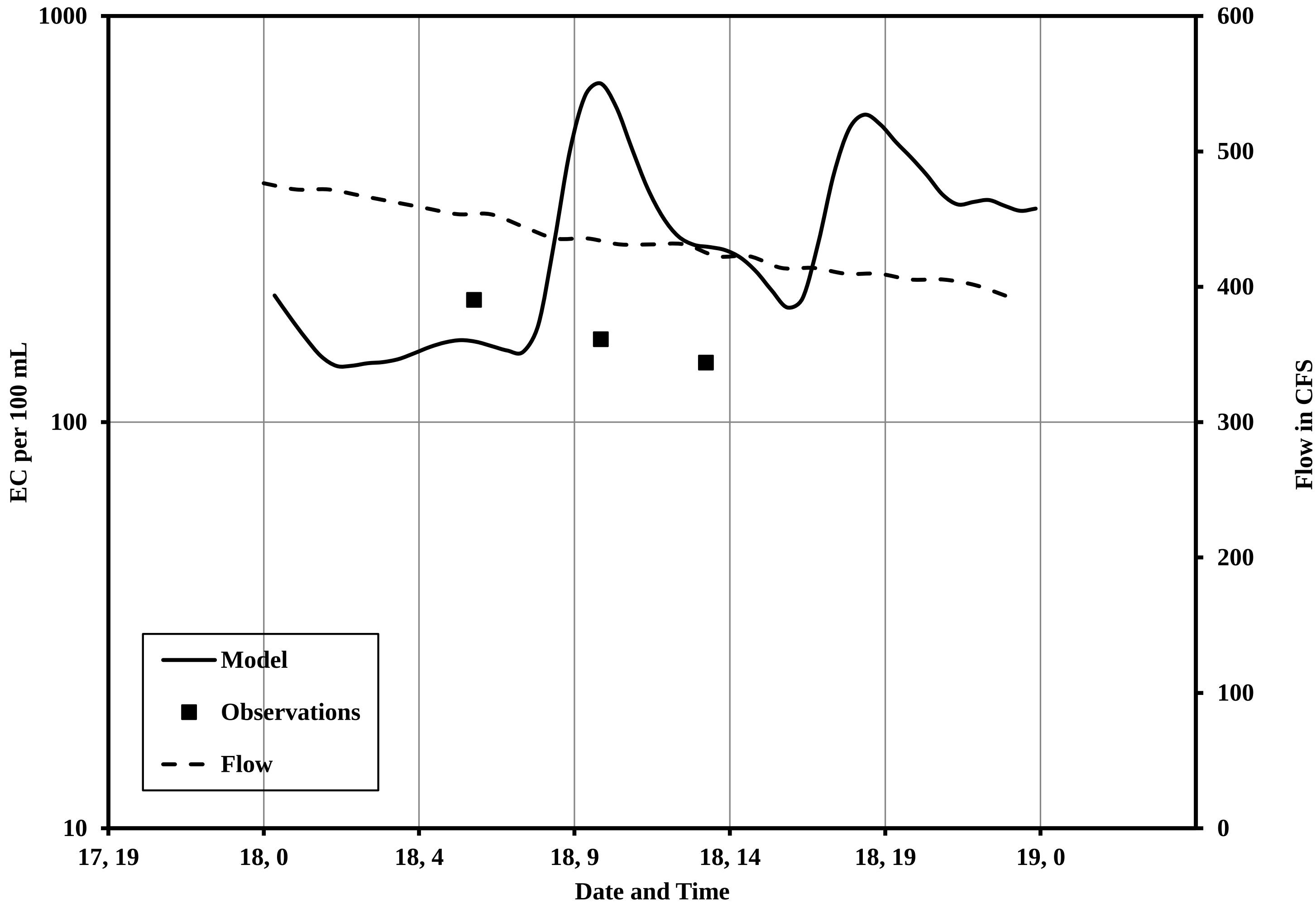

Figure VIII-36 - MORLOT DRY EVENT 2 AUGUST 18, 2009

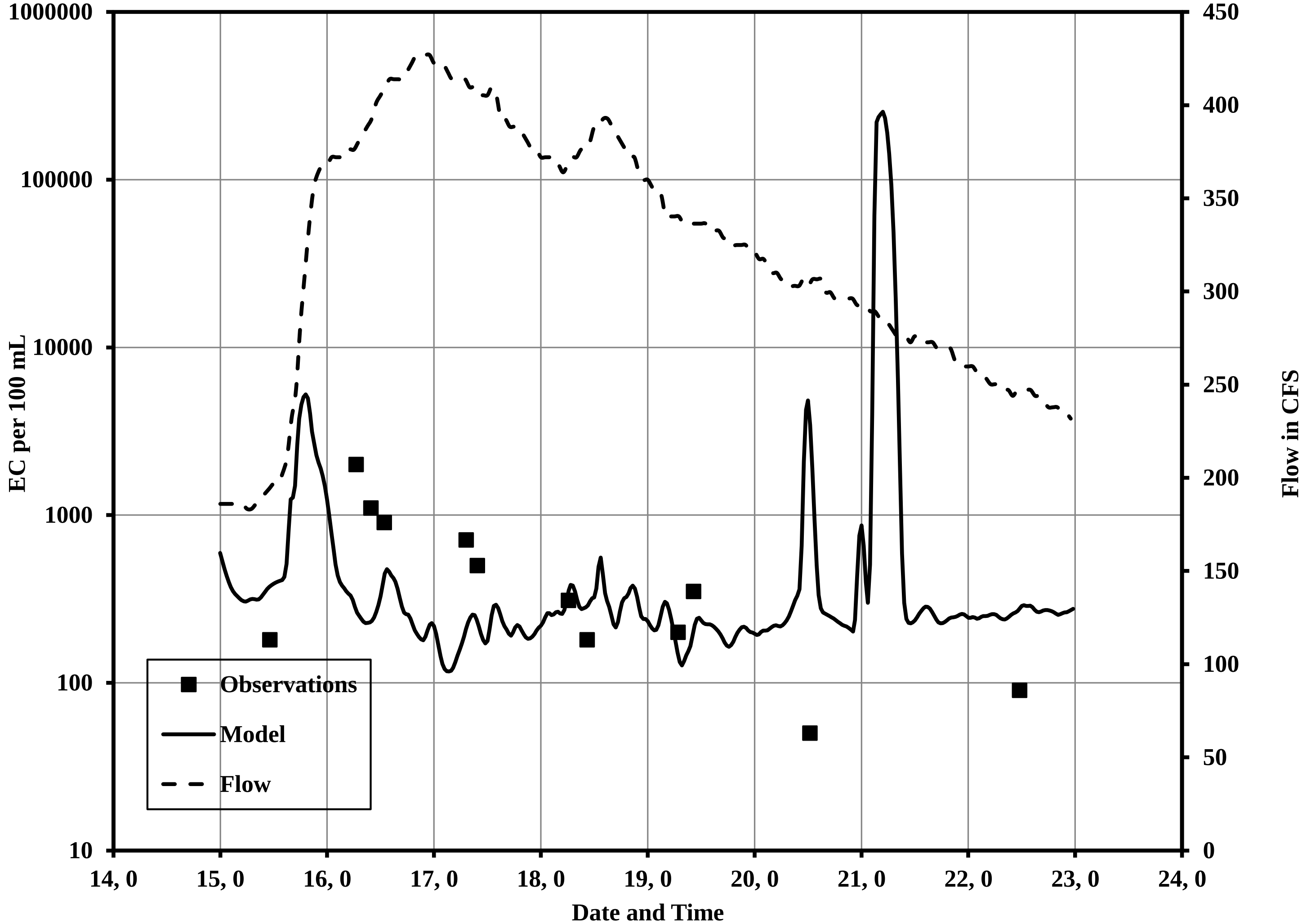

Figure VIII-37 - MORLOT WET EVENT 1 WITH FLOW OCTOBER 15-22, 2009

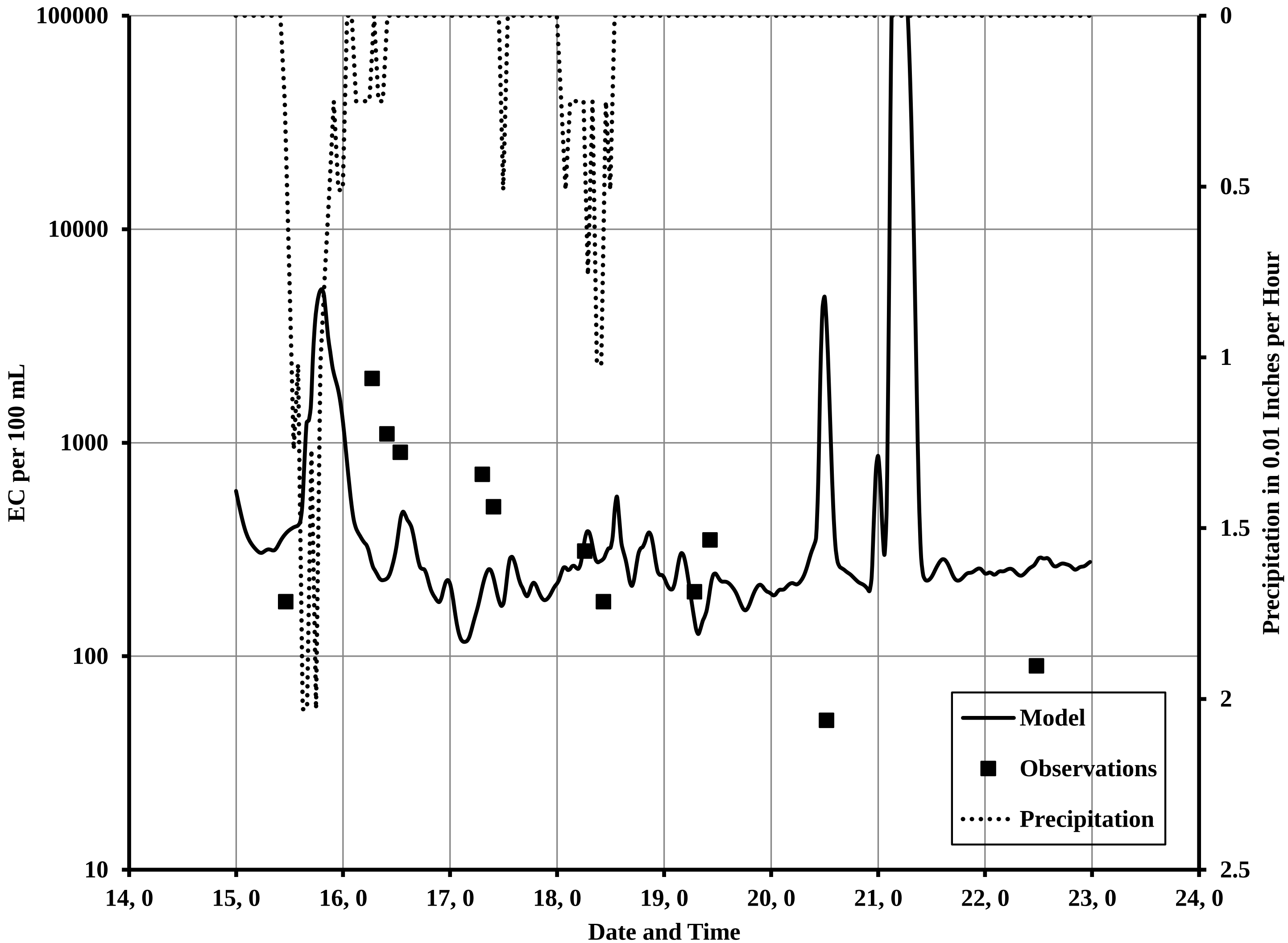

Figure VIII-38 - MORLOT WET EVENT 1 WITH PRECIPITATION OCTOBER 15-22, 2009

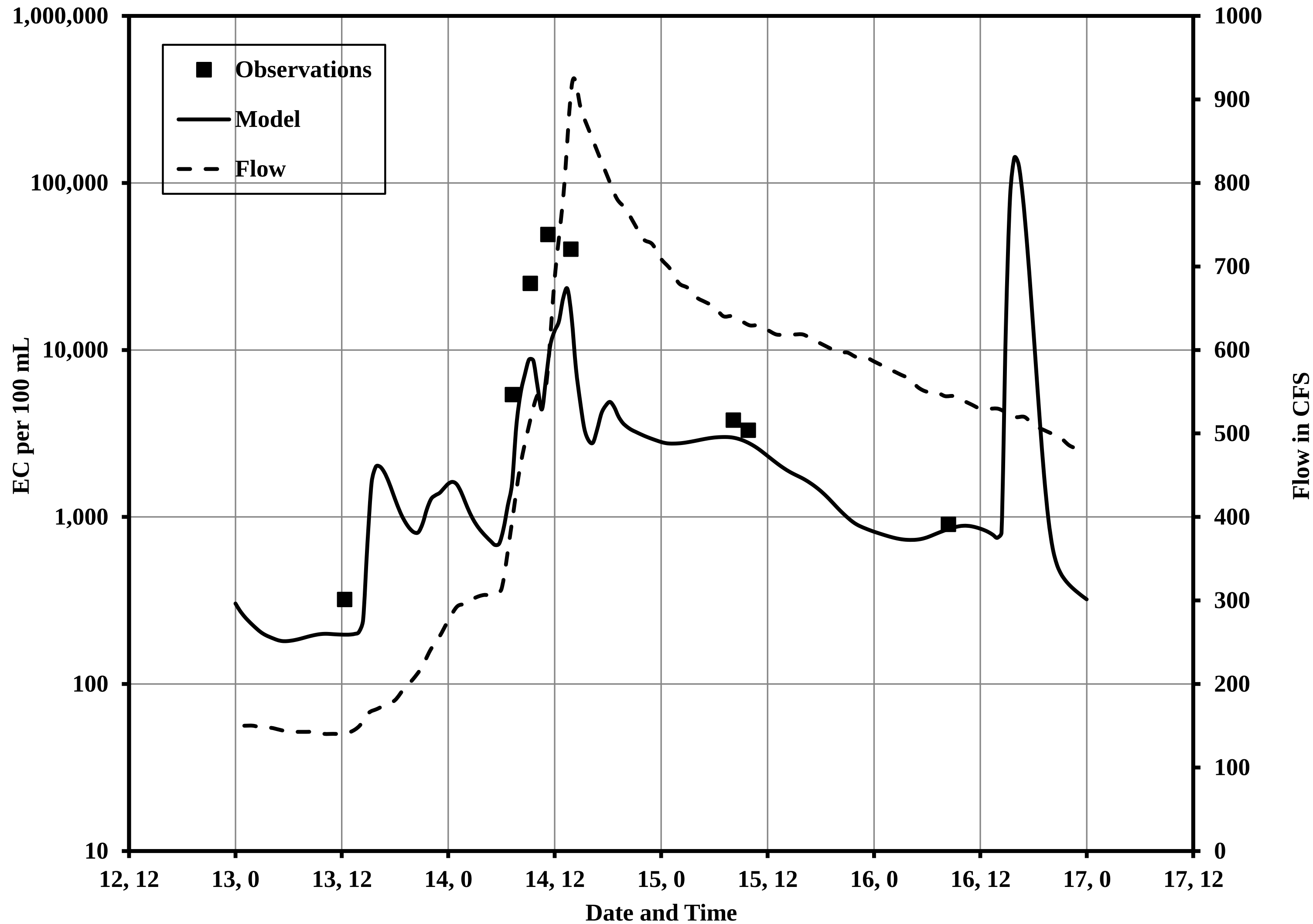

Figure VIII-39 - MORLOT WET EVENT 2 WITH FLOW JULY 13-16, 2010

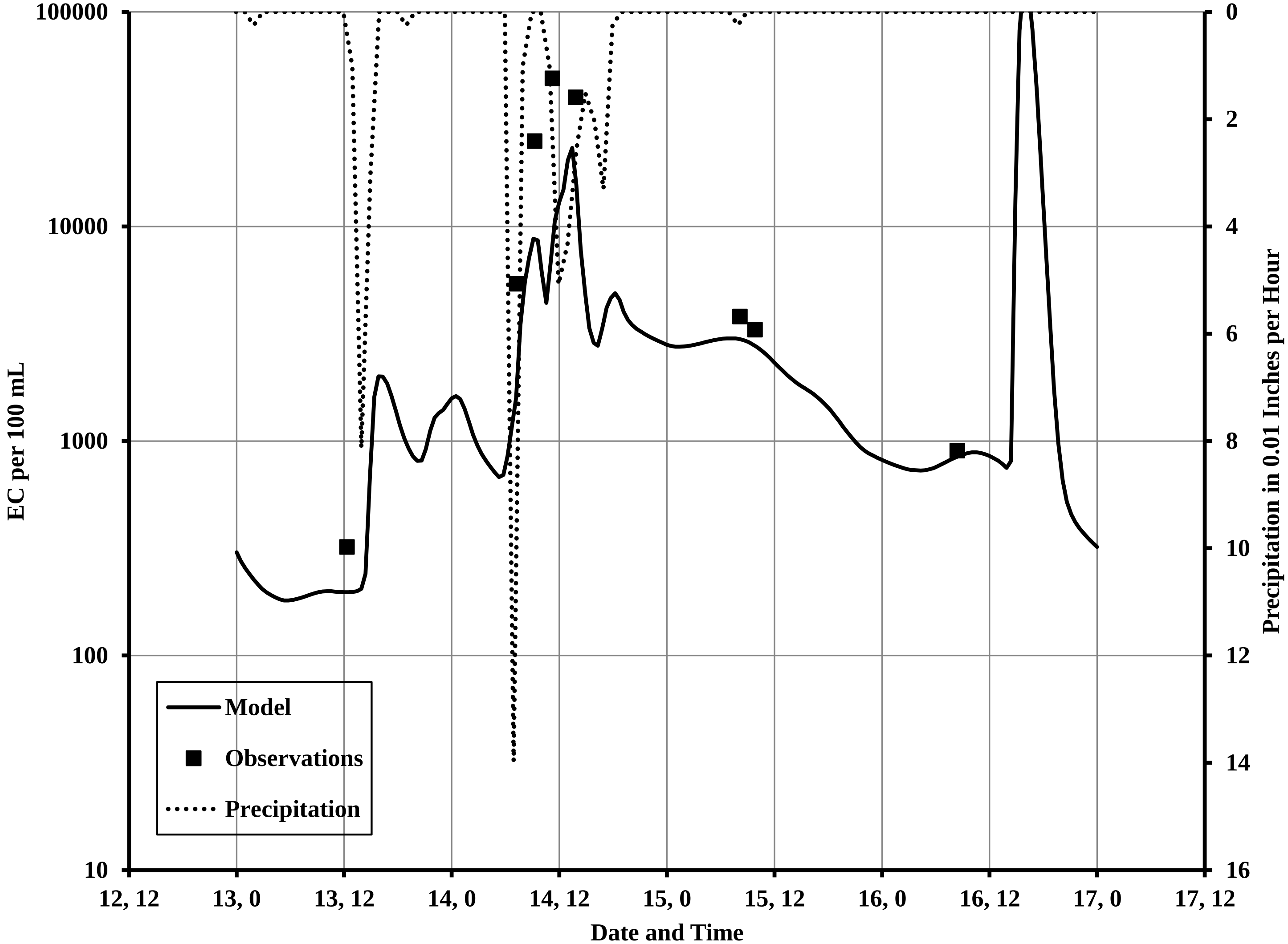

Figure VIII-40 - MORLOT WET EVENT 2 WITH PRECIPITATION JULY 13-16, 2010

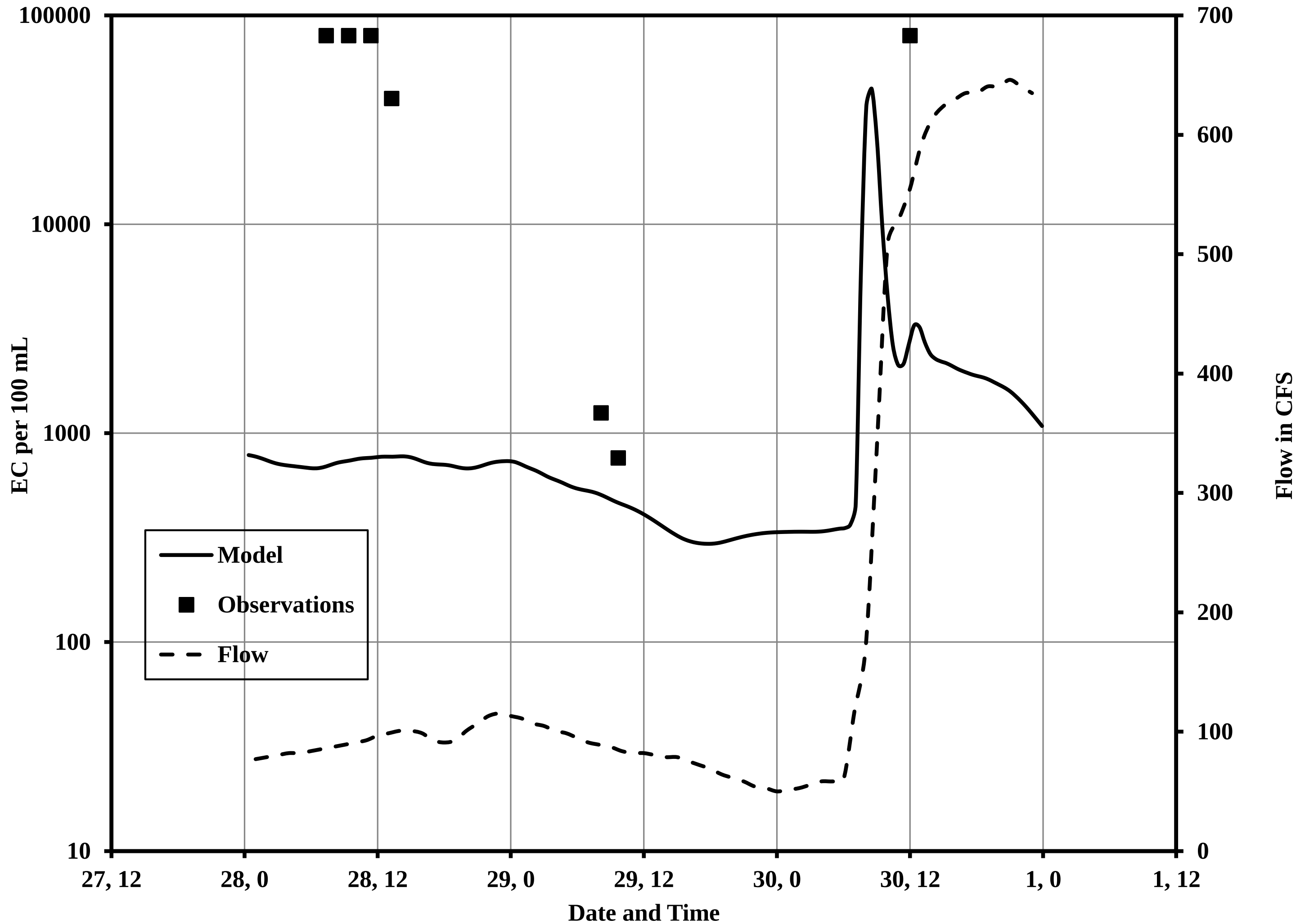

Figure VIII-41 - MORLOT WET EVENT 3 WITH FLOW OCTOBER 15, 28-30, 2010

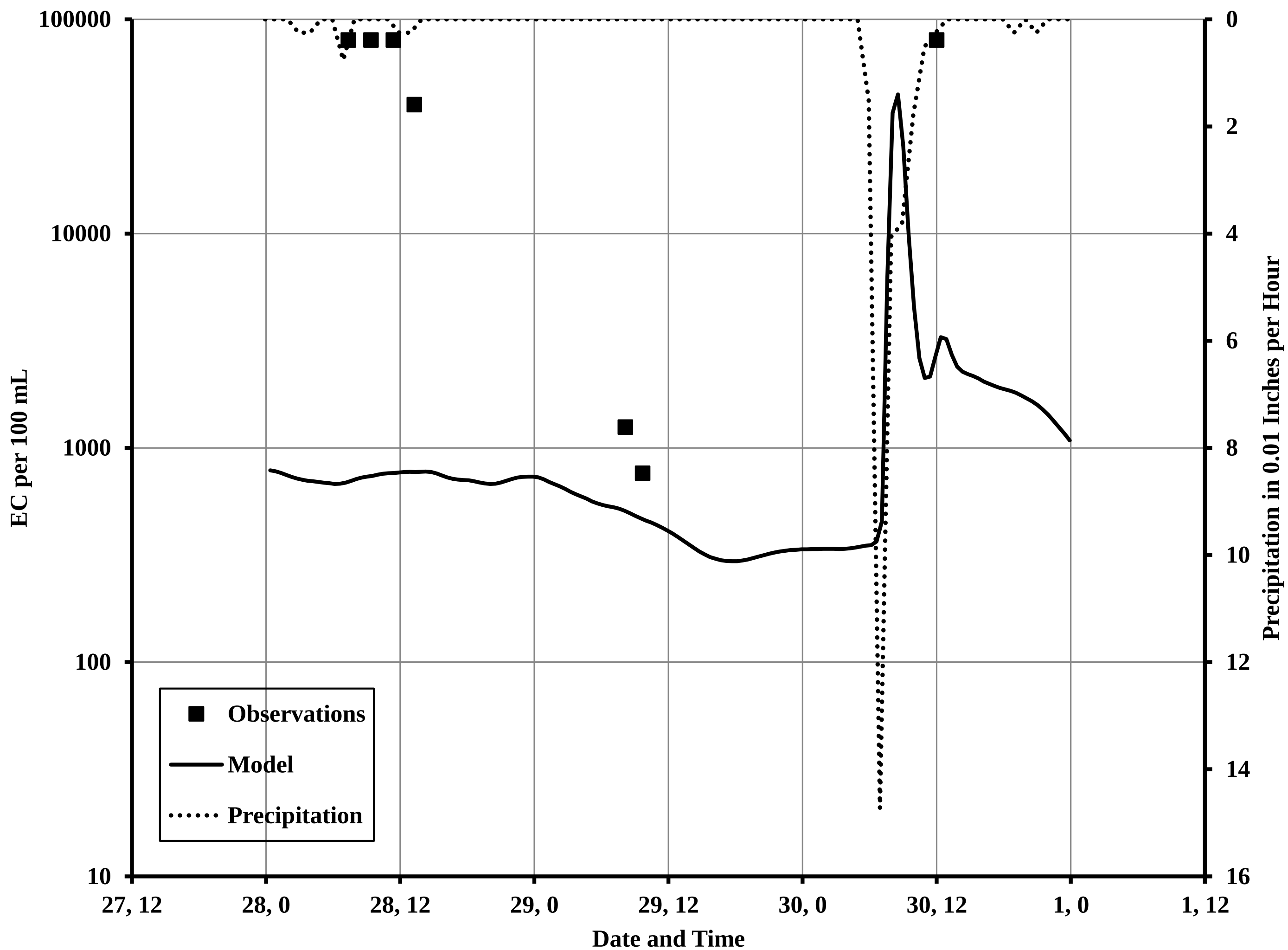

Figure VIII-42 - MORLOT WET EVENT 3 WITH PRECIPITATION OCTOBER 15, 28-30, 2010

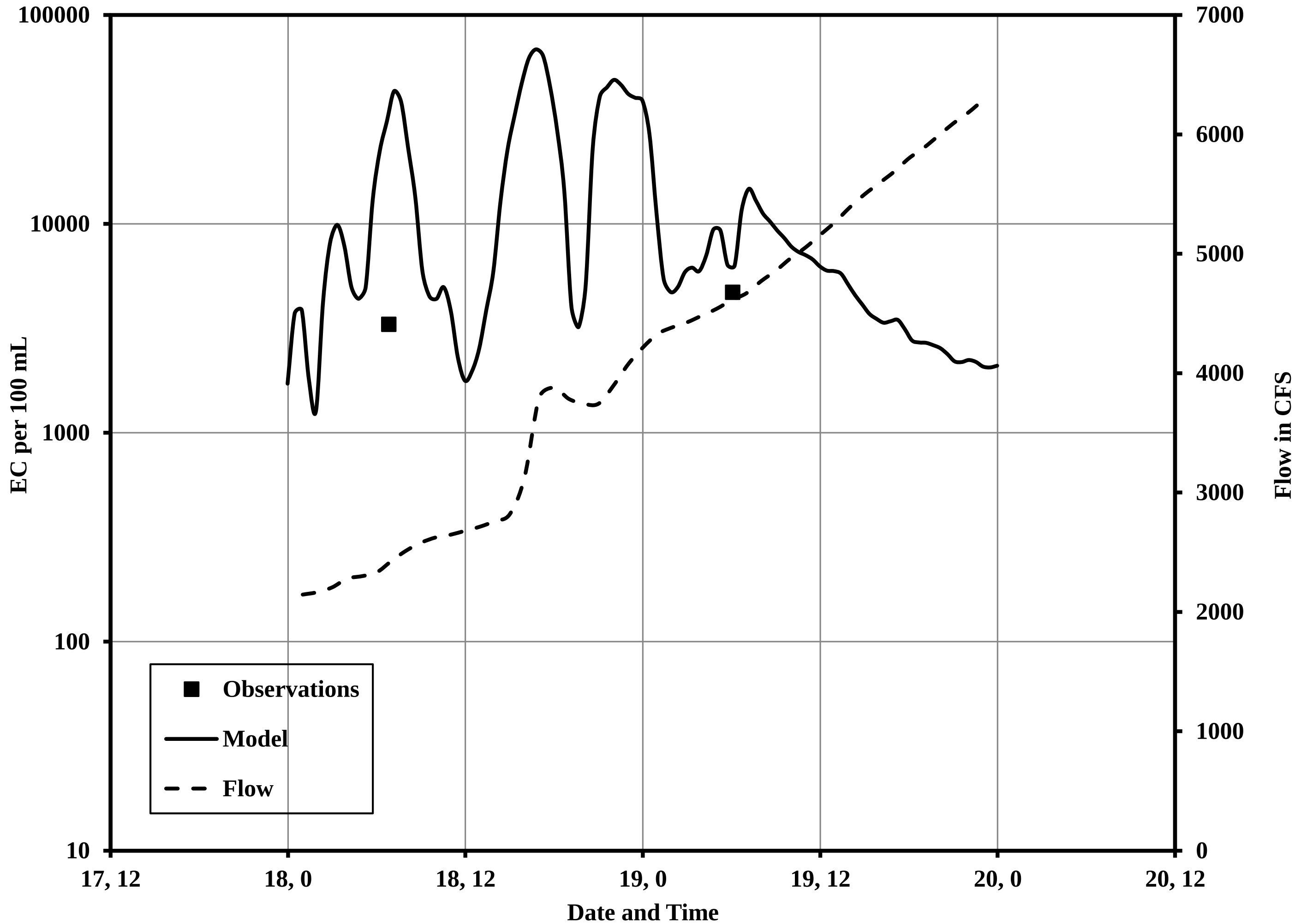

Figure VIII-43 - MORLOT WET EVENT 4&5 WITH FLOW MAY 18-19, 2011

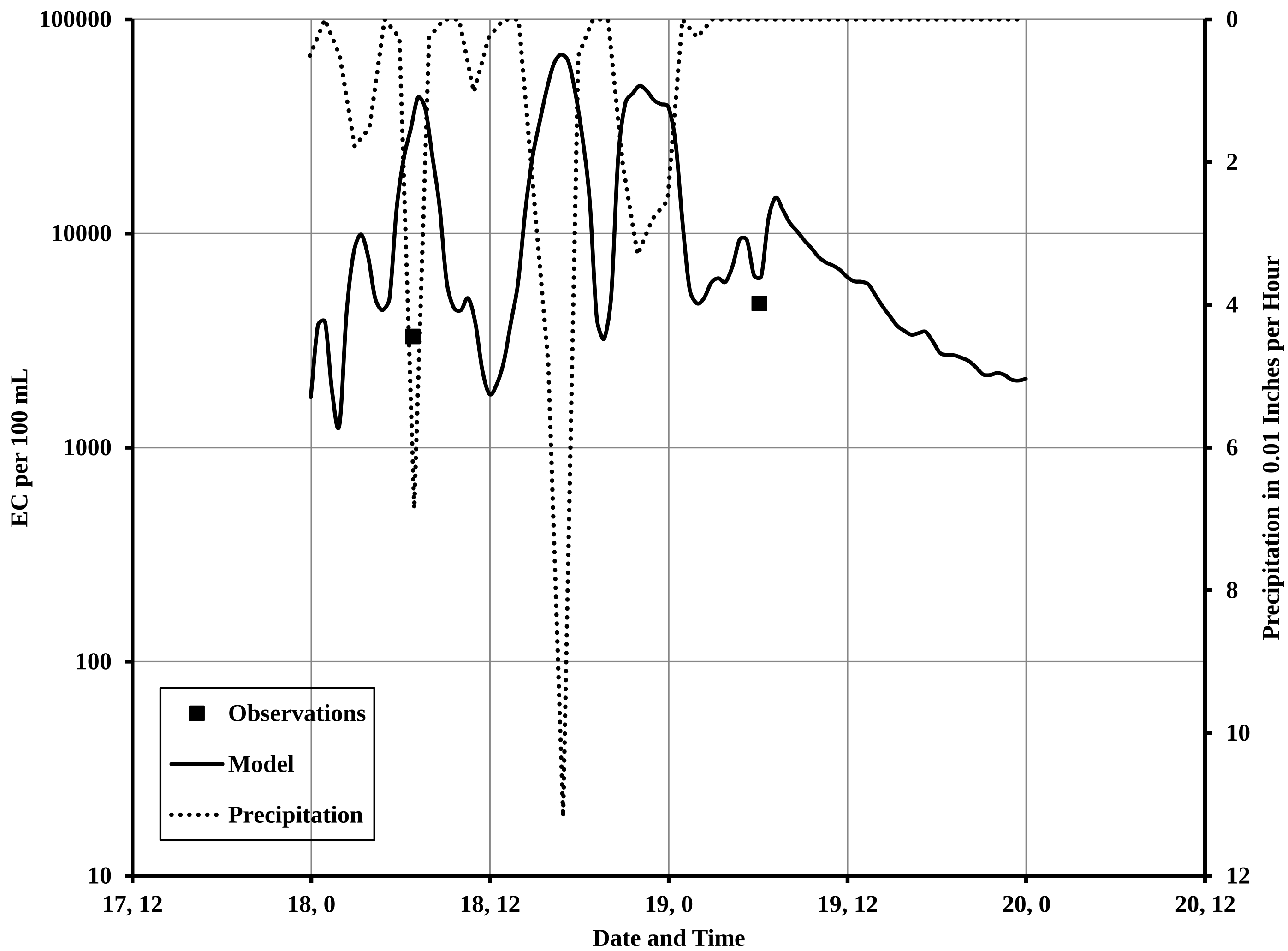

Figure VIII-44 - MORLOT WET EVENT 4&5 WITH PRECIPITATION MAY 18-19, 2011

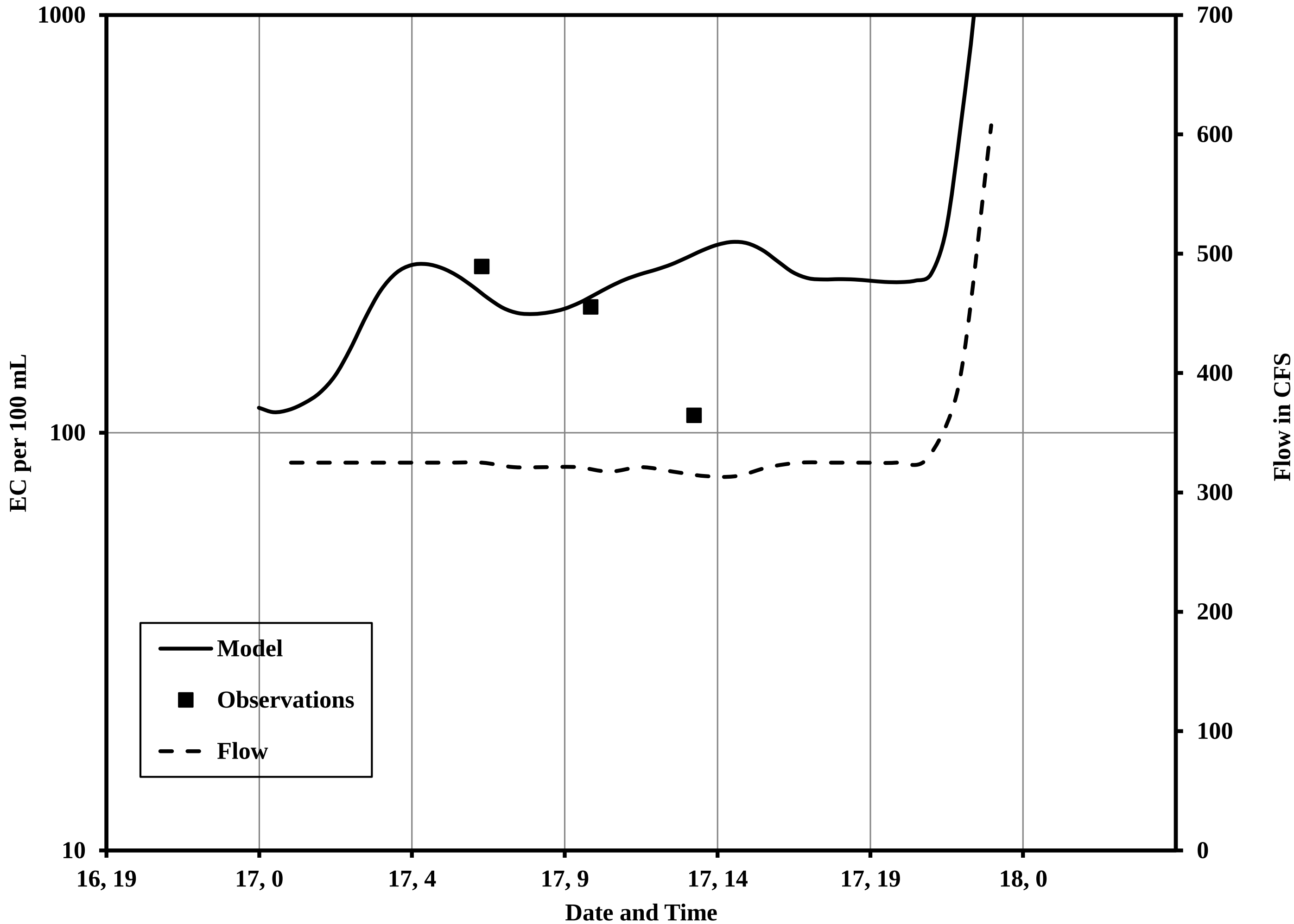

Figure VIII-45 - MARKET DRY EVENT 1 JULY 17, 2009

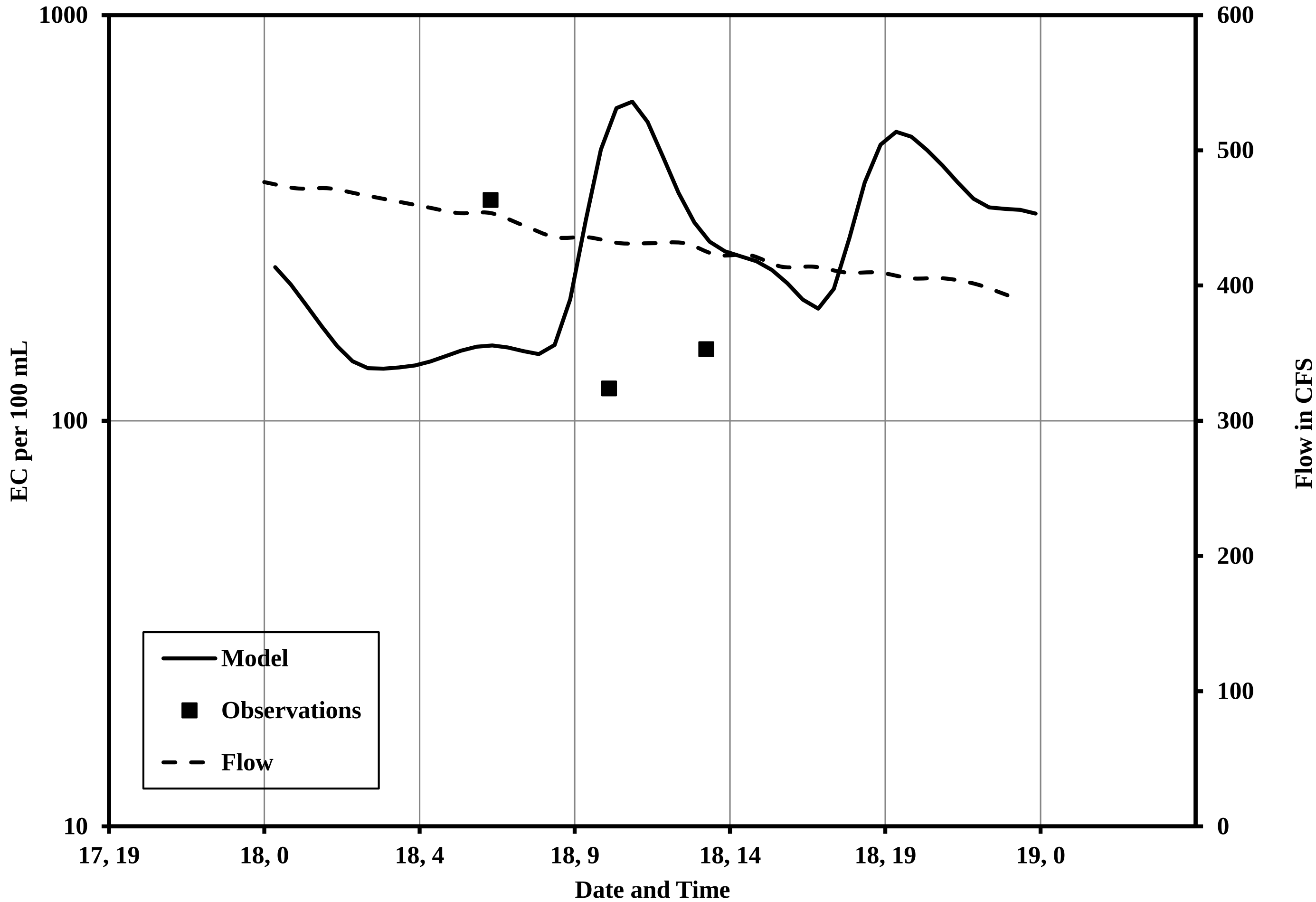

Figure VIII-46 - MARKET DRY EVENT 2 AUGUST 18, 2009

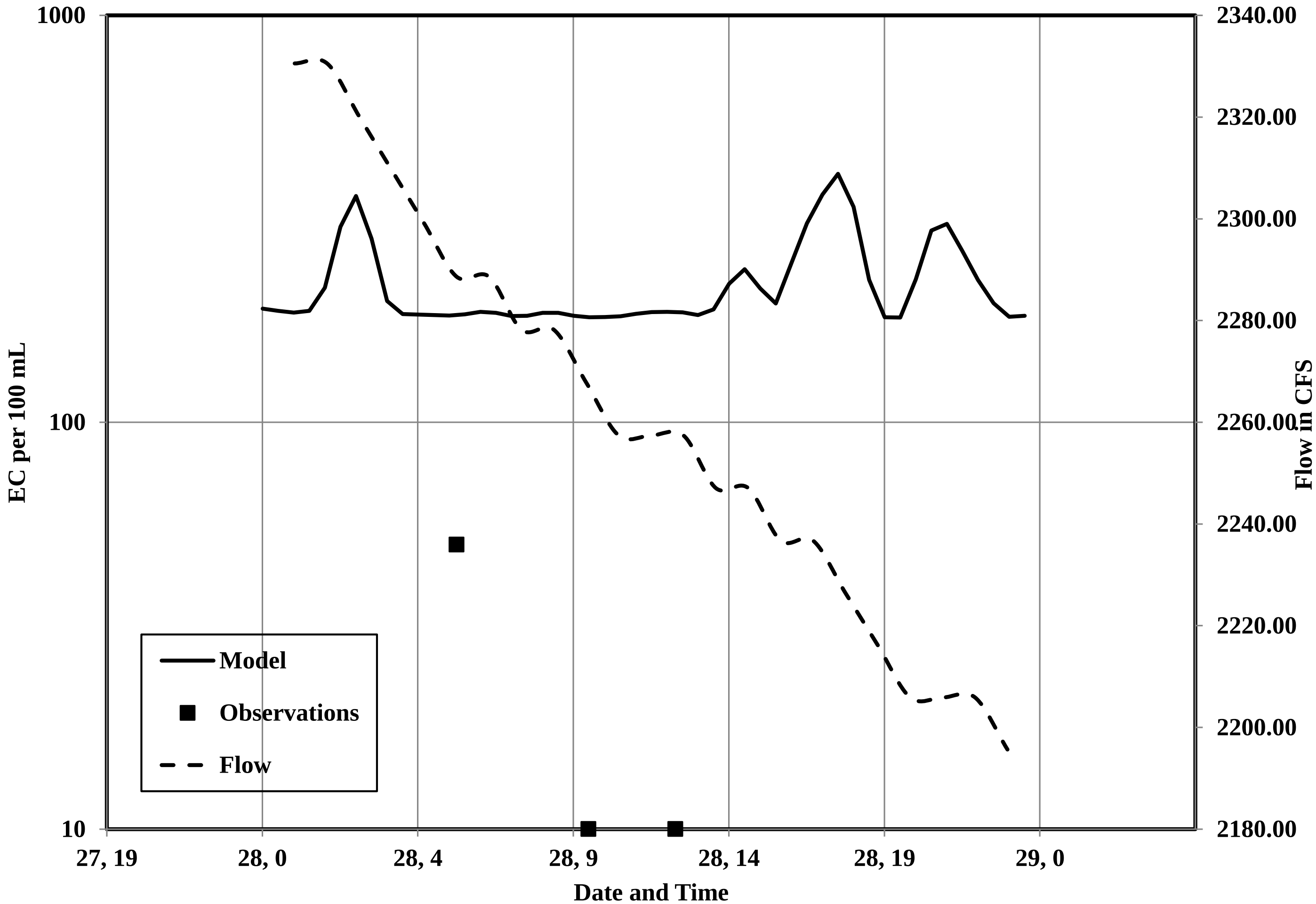

Figure VIII-47 - MARKET DRY EVENT 3 MARCH 28, 2011

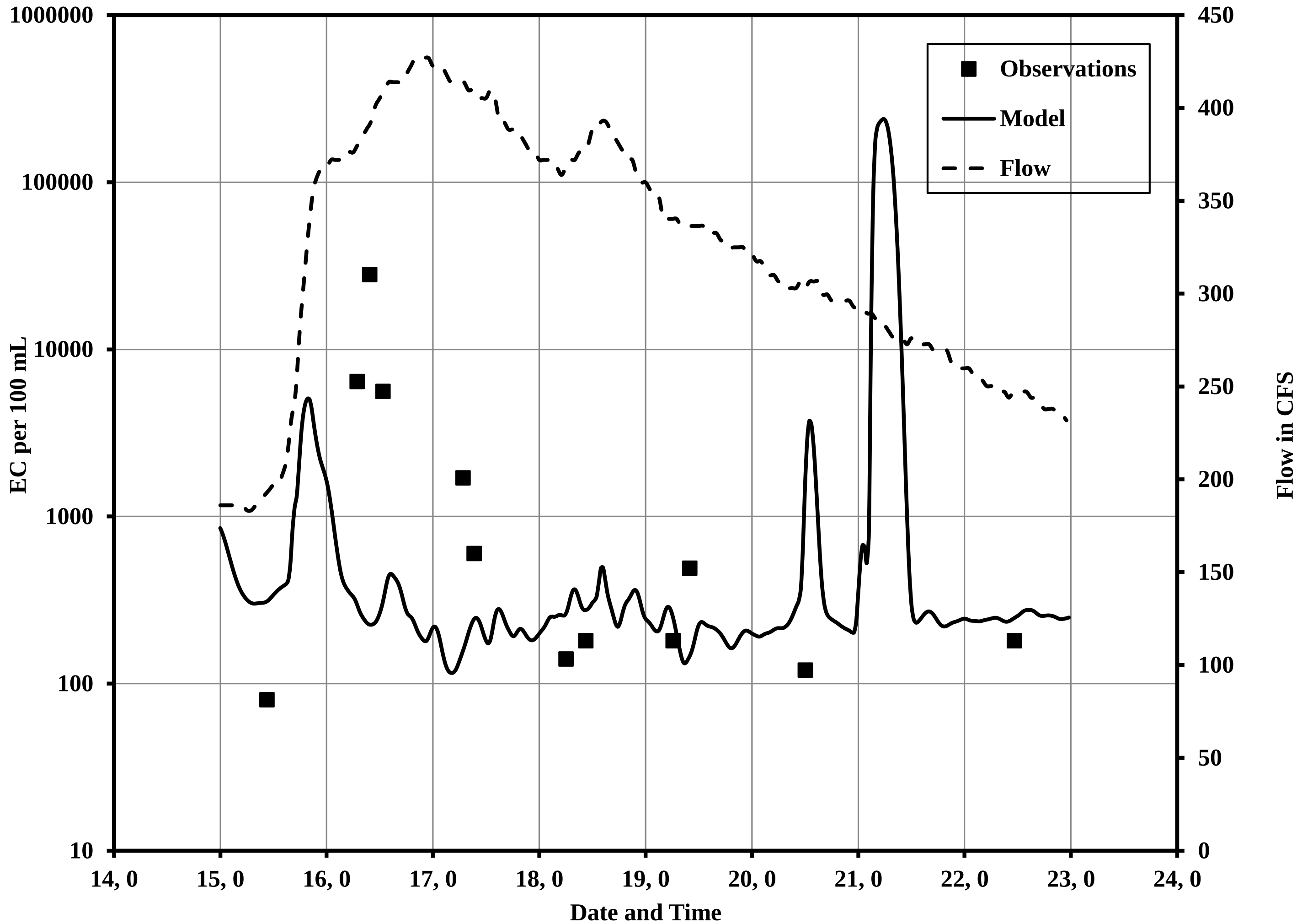

Figure VIII-48 - MARKET WET EVENT 1 WITH FLOW OCTOBER 15-22, 2009

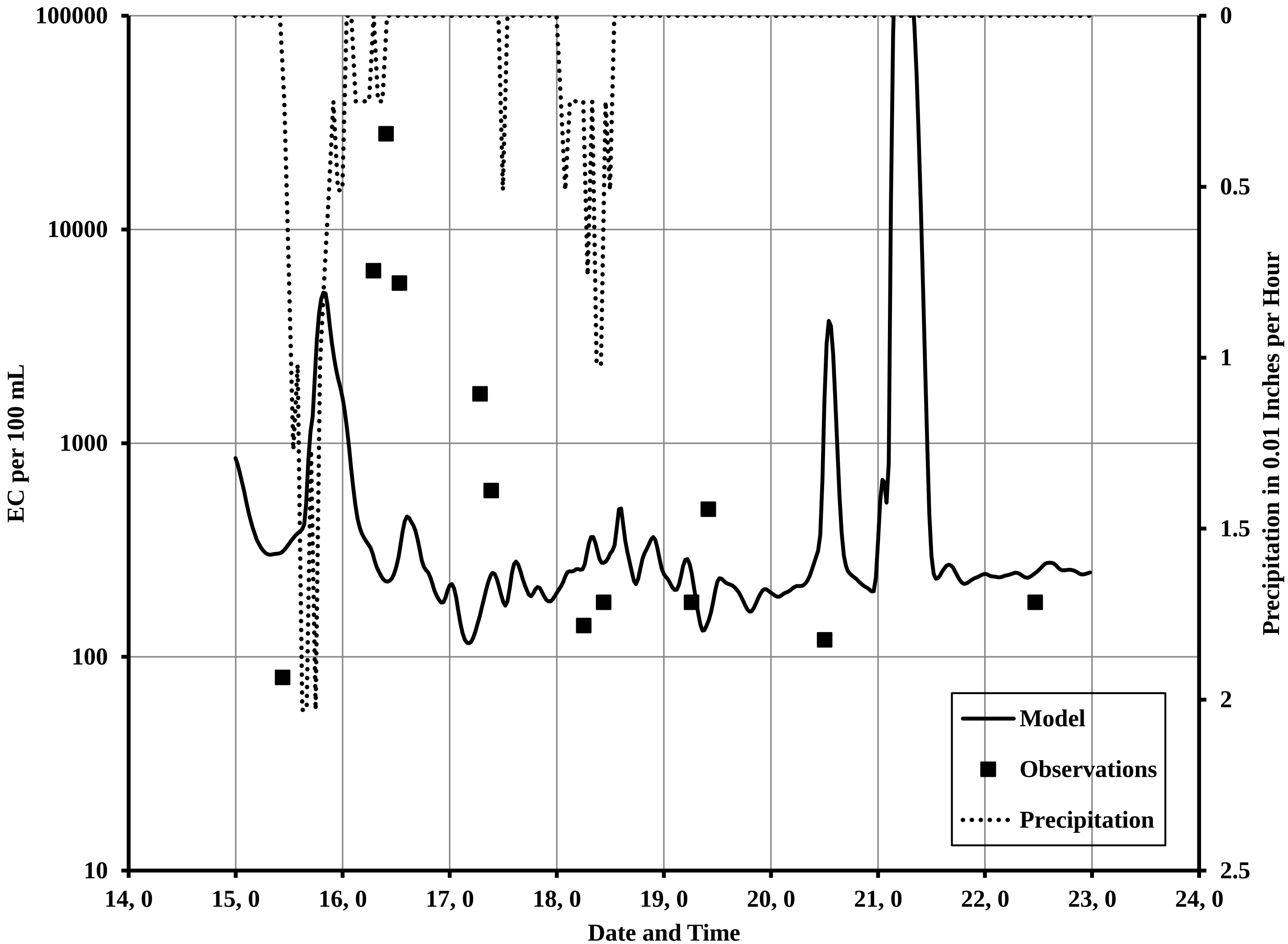

Figure VIII-49 - MARKET WET EVENT 1 WITH PRECIPITATION OCTOBER 15-22, 2009

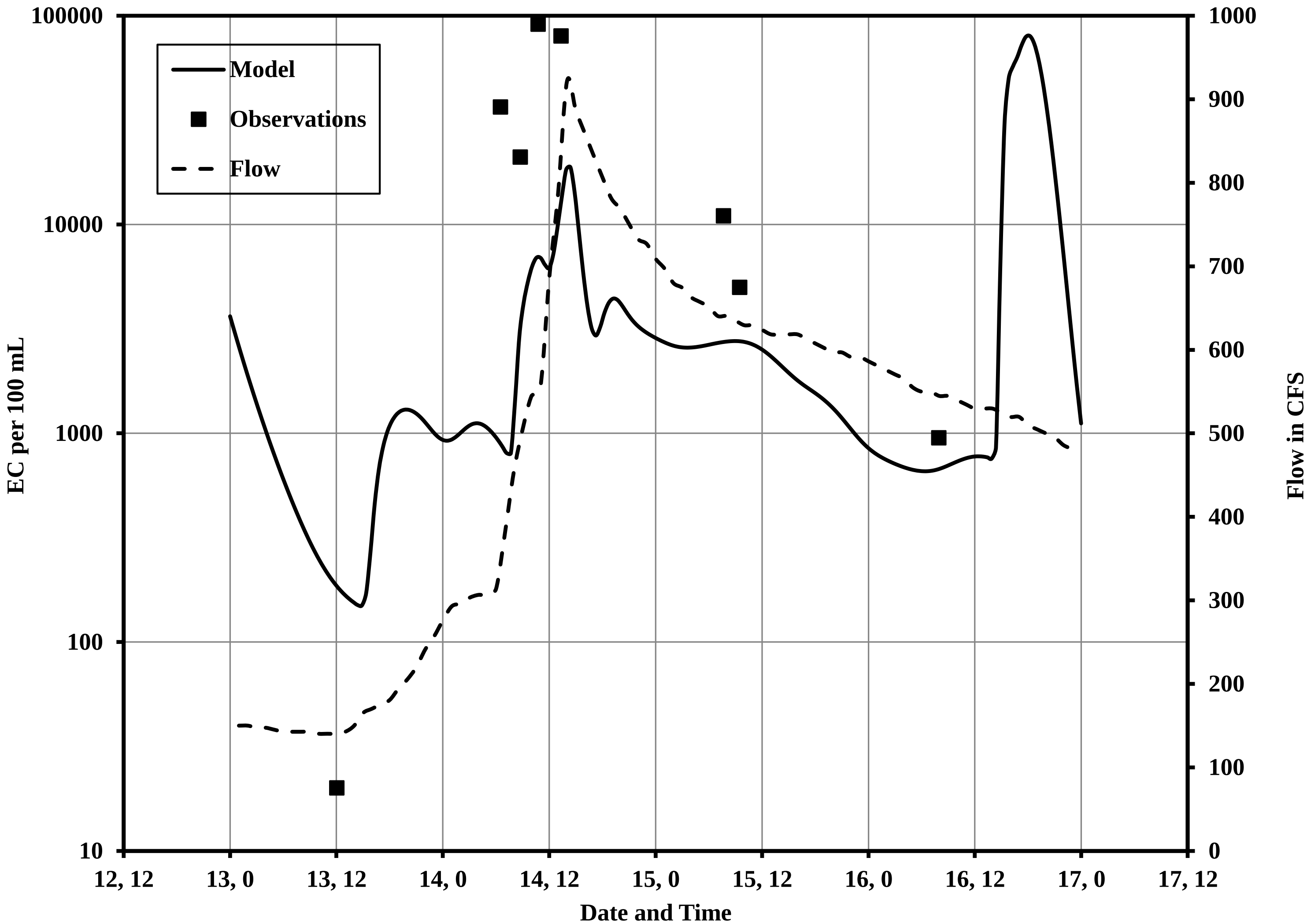

Figure VIII-50 - MARKET WET EVENT 2 WITH FLOW JULY 13-16, 2010

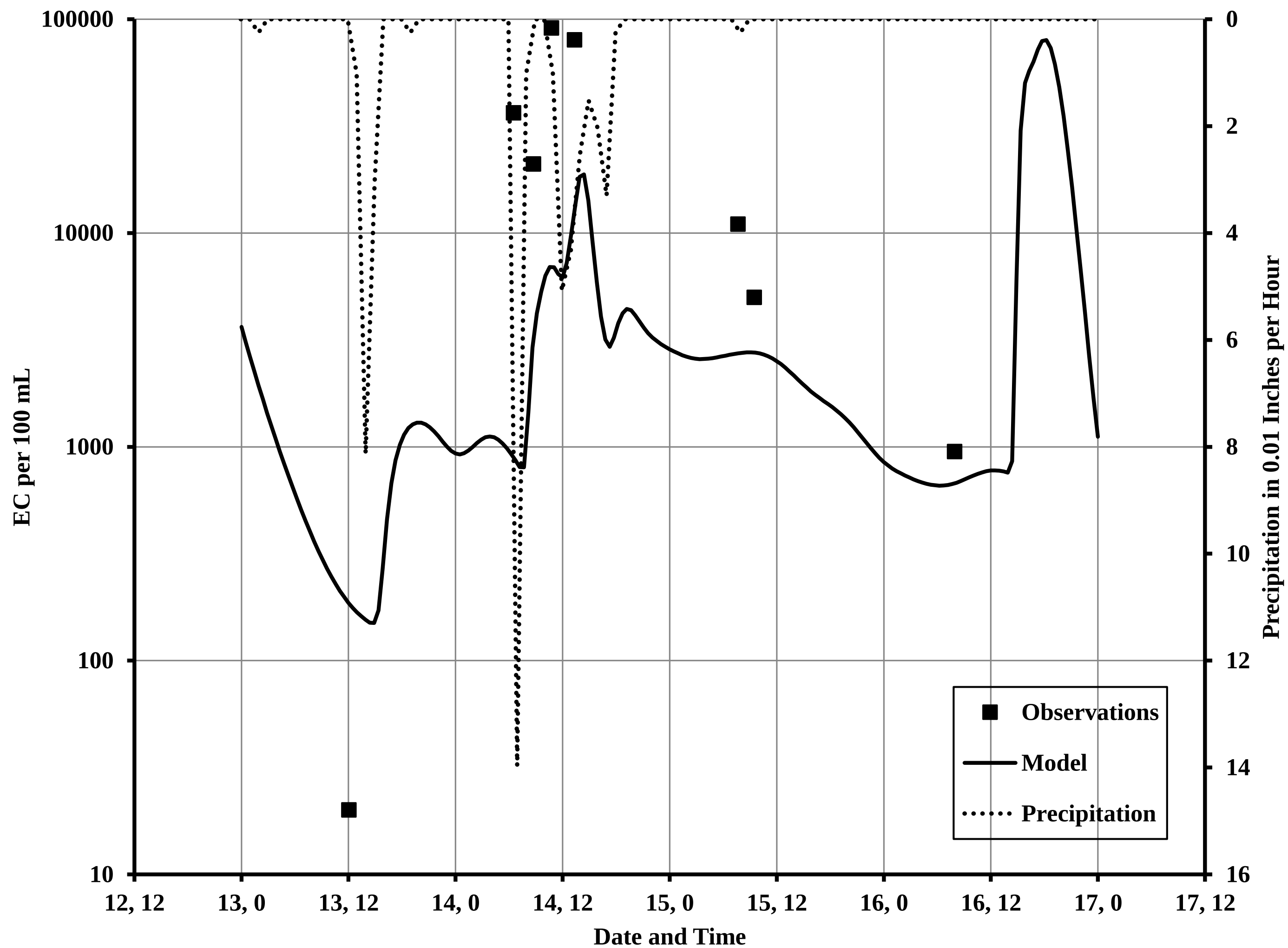

Figure VIII-51 - MARKET WET EVENT 2 WITH PRECIPITATION JULY 13-16, 2010

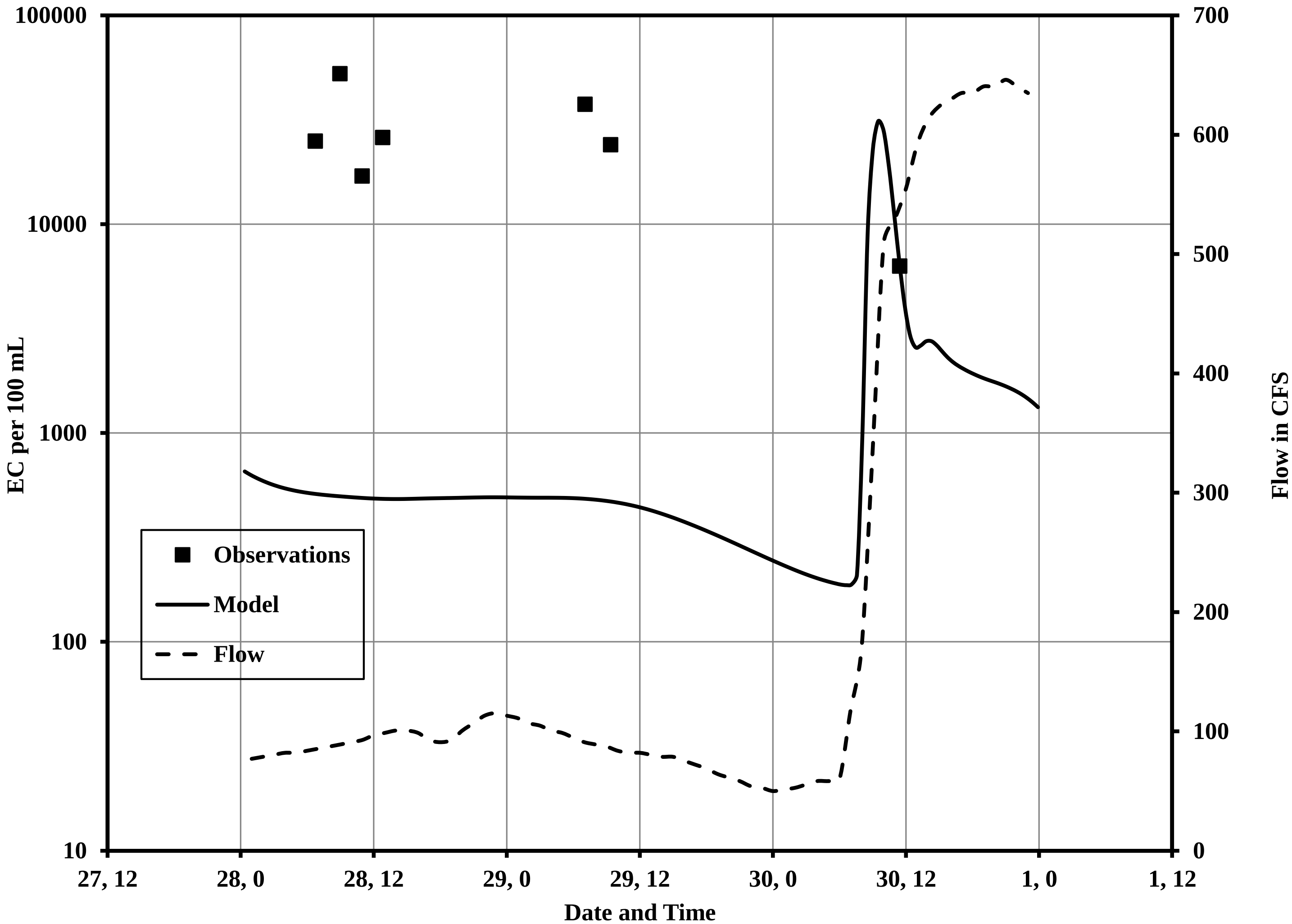

Figure VIII-52 - MARKET WET EVENT 3 WITH FLOW OCTOBER 15, 28-30, 2010

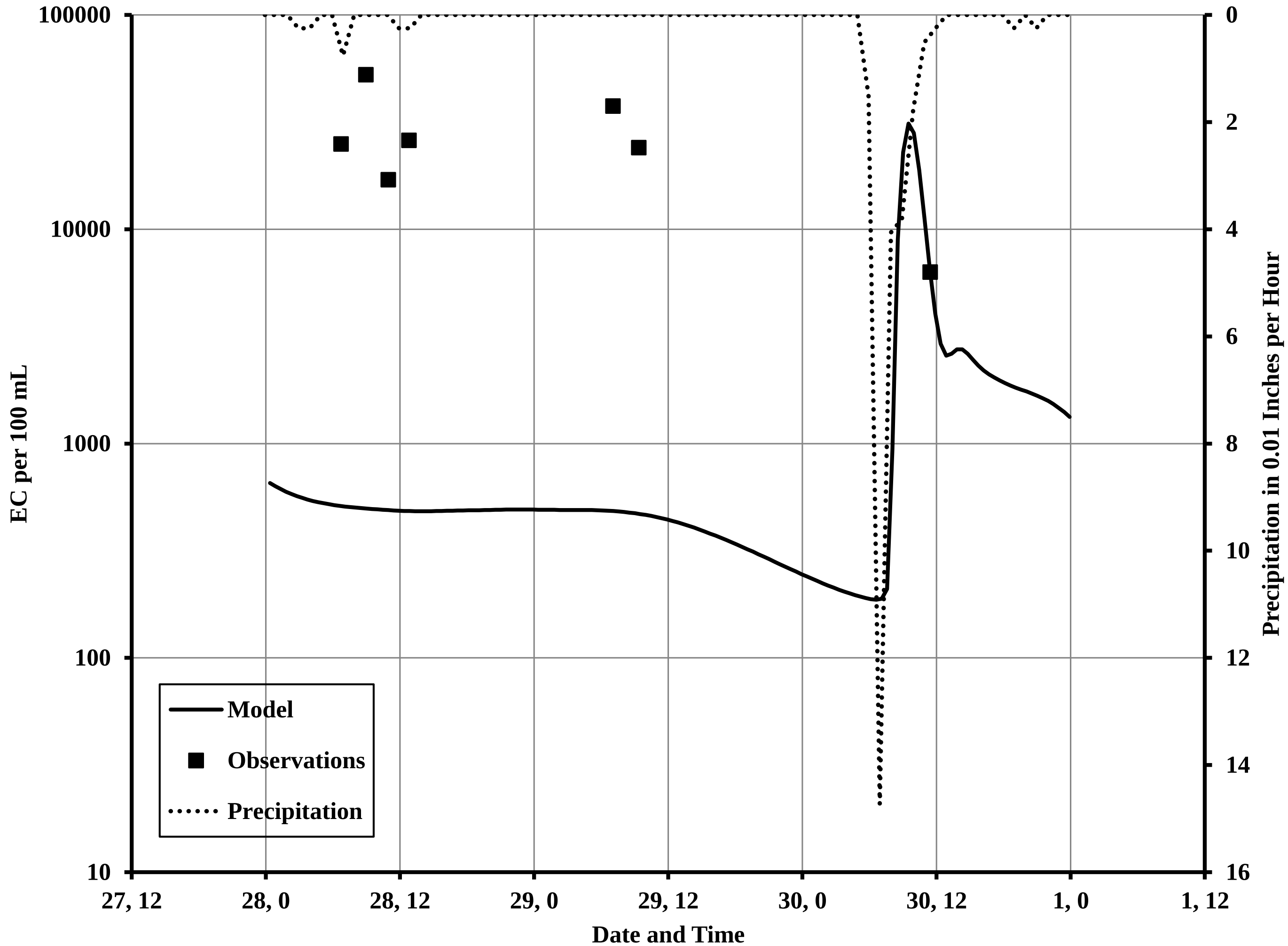

Figure VIII-53 - MARKET WET EVENT 3 WITH PRECIPITATION OCTOBER 15, 28-30, 2010

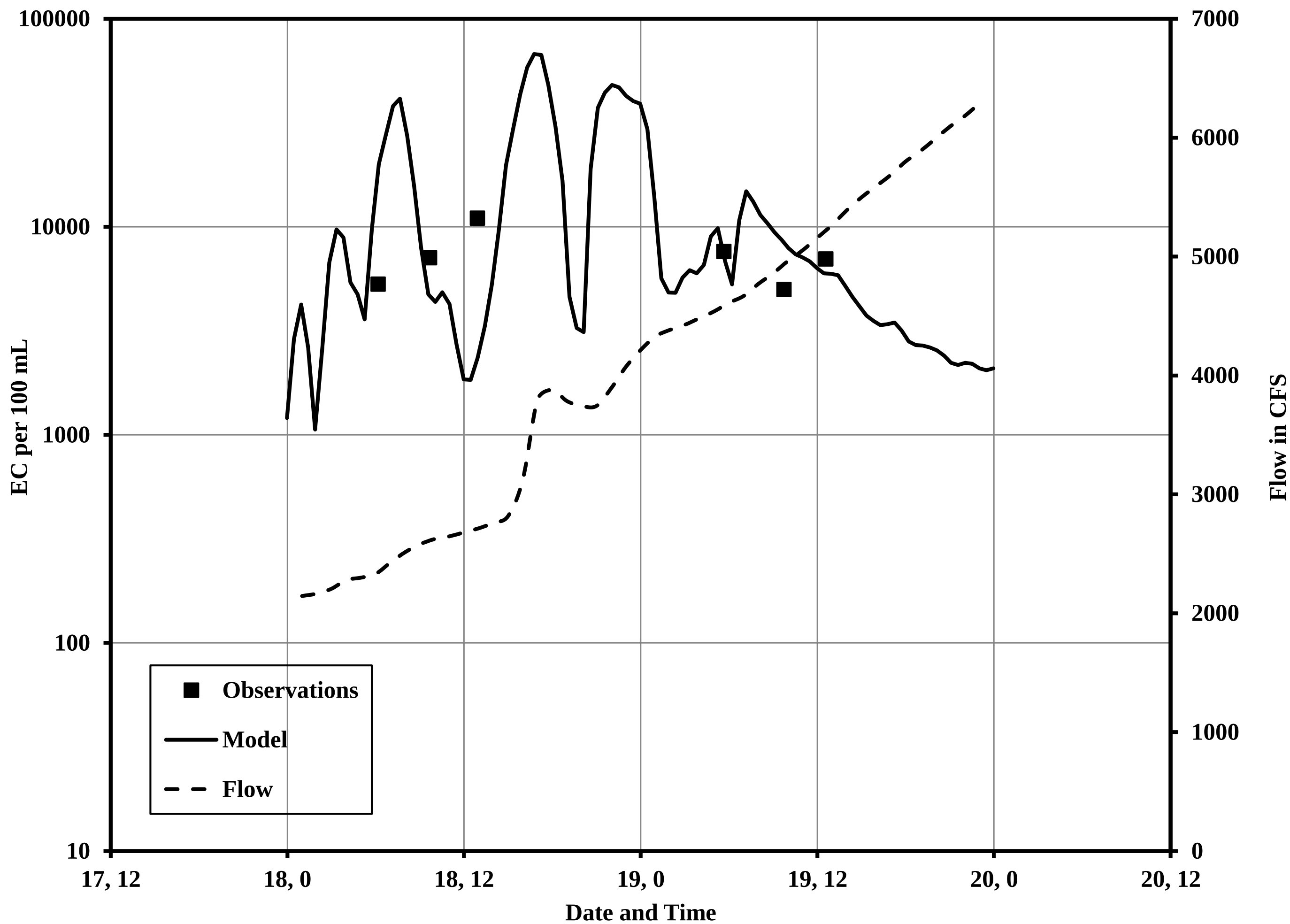

Figure VIII-54 - MARKET WET EVENT 4&5 WITH FLOW MAY 18-19, 2011

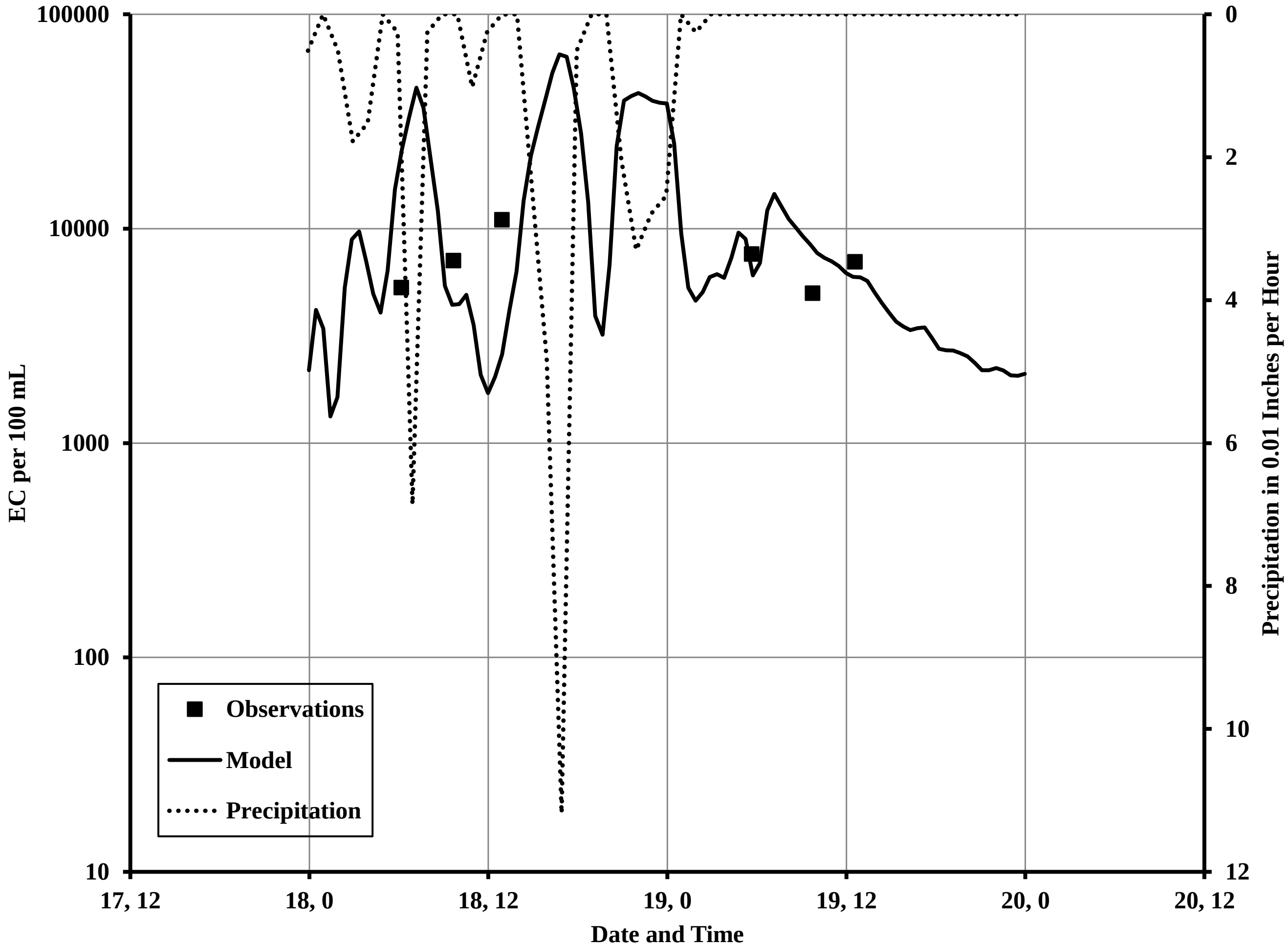

Figure VIII-55 - MARKET WET EVENT 4&5 WITH PRECIPITATION MAY 18-19, 2011

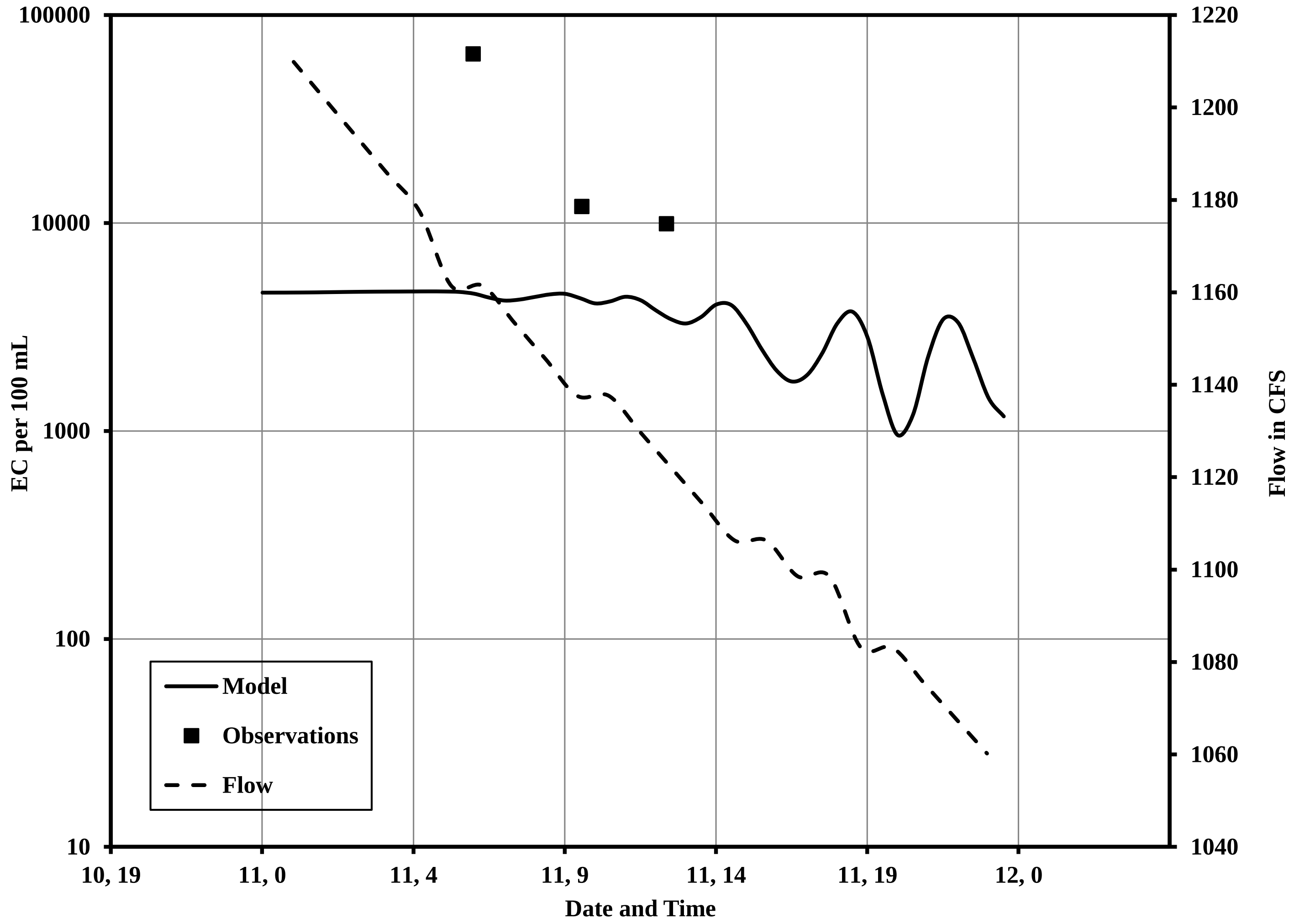

Figure VIII-56 - MARKET WET EVENT 6 WITH FLOW OCTOBER 11, 2011